Appendix E

Preliminary Geotechnical and Geological Hazards Evaluation

REPORT

PRELIMINARY GEOTECHNICAL AND
GEOLOGIC HAZARDS EVALUATION
SOLAR ONE PROJECT
SAN BERNARDINO COUNTY,
CALIFORNIA

PREPARED FOR:

STIRLING ENERGY SYSTEMS, INC.

URS PROJECT NO. 27658173.10000

NOVEMBER 14, 2008

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REPORT

PRELIMINARY GEOTECHNICAL AND GEOLOGIC HAZARDS EVALUATION SOLAR ONE PROJECT SAN BERNARDINO COUNTY, CALIFORNIA

Prepared for

Stirling Energy Systems, Inc.
2920 E. Camelback Road, Suite 150
Phoenix, AZ 85016

URS Project No. 27658173.10000

November 14, 2008

URS

1615 Murray Canyon Road, Suite 1000
San Diego, CA 92108-4314
619.294.9400 Fax: 619.293.7920

November 14, 2008

Mr. John Egan
Stirling Energy Systems, Inc.
2920 E. Camelback Road, Suite 150
Phoenix, AZ 85016

Subject: Preliminary Geotechnical and Geologic Hazards Evaluation
Solar One Project
San Bernardino County, California
URS Project No. 27658173.10000

Dear Mr. Egan:

URS Corporation Americas (URS) is pleased to submit the following report presenting the results of our preliminary geotechnical and geologic hazards evaluation for the proposed Solar One Project. This investigation was performed in general accordance with our scope of services dated October 16, 2008.

This report presents our initial findings and preliminary conclusions regarding geotechnical issues and geologic hazards at the proposed site. The recommendations are based on limited geologic reconnaissance, mapping and research. No geotechnical field exploration or laboratory testing has been performed. The results of the study indicate that the site should be suitable for the proposed development, provided the geotechnical and geologic considerations discussed in this report are incorporated into the planning and design. Site-specific geotechnical investigation will be required to support final design.

We are pleased to be part of this important project, and if you have any questions, please contact us at (619) 294-9400.

Sincerely,

URS CORPORATION

Michael E. Hatch, C.E.G. 1925
Associate Geologist

Kelly C. Giesing, G.E. 2749
Project Engineer

Derek Rector, P.G., 8406
Assistance Project Geologist

MEH/KCG/DR:ml

URS Corporation
1615 Murray Canyon Road, Suite 1000
San Diego, CA 92108
Tel: 619.294.9400
Fax: 619.293.7920

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AFC	Application for Certification
BNSF	Burlington Northern Santa Fe Railway
CBC	California Building Code
CEC	California Energy Commission
CIDH	cast-in-drilled hole
ECSZ	Eastern California Shear Zone
GPS	global positioning system
kV	kiloVolt
mm/year	millimeters per year
msl	Mean Sea Level
MW	Megawatt
M_w	Moment Magnitude
Project	Solar One Project
SES	Stirling Energy Systems, Inc.
SPT	Standard Penetration Test
URS	URS Corporation Americas

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SECTION 1 INTRODUCTION

This report presents the results of URS Corporation Americas' (URS) preliminary geotechnical and geologic hazards evaluation for the proposed Solar One Project (Project). The site is located in San Bernardino County, about 37 miles east of Barstow, California. The location of the site is shown on the Vicinity Map, Figure 1.

Stirling Energy Systems, Inc. (SES) is considering the site for development as a solar-powered electrical generation station. This preliminary evaluation was undertaken to support SES in their Application for Certification (AFC) to the California Energy Commission (CEC) and to provide project planning and preliminary engineering design information. The evaluation was performed at a reconnaissance level; field geotechnical investigations will be required to support final design.

1.1 PROJECT DESCRIPTION

The Project will encompass approximately 5,000 acres. The Project boundary and major features are shown on the Site Plan, Figure 2. The site is bounded by Interstate 40 on the south, an existing transmission line and Pisgah Substation on the east and the Cady Mountains on the north. The majority of the Project components are east of Hector Road, although the site extends approximately 2 miles west of Hector Road on the south side of the Project area. An existing Burlington Northern Santa Fe (BNSF) railroad line runs approximately east-west through the site.

The Project will be constructed in two stages: a 500-megawatt (MW) stage and a 350-MW stage. Power will be supplied by up to 34,000 SunCatchers, which are individual solar dish structures each supported on a single metal fin-pipe pile foundation that is vibrated into the ground. These foundations are expected to be 10 to 15 feet long and 24 inches in diameter, with 12-inch wide fins extending from four sides of the pipe pile. Drilled pier foundations (also called cast-in-drilled hole [CIDH] piles) would be used where fin-pipe foundations are not practical. SunCatcher foundations will be installed at a spacing of approximately 112 feet in the east-west direction and 56 feet in the north-south direction. The dish foundations will be lightly loaded, with uplift or overturning forces expected to control design considerations.

A 42-acre Main Services Complex will be constructed near the center of the site and will include three SunCatcher assembly buildings, administrative, operational and maintenance facilities, and wastewater treatment and stormwater retention basins. Preliminary details of the structures in the Main Services Complex are as follows:

- Administration/control building - one story, approximately 200 feet long by 100 feet wide by 14 feet high;
- Maintenance building - approximately 250 feet long by 180 feet wide by 44 feet high;
- Control room – approximately 100 feet long by 50 feet wide;
- Three assembly buildings – each 211 feet long by 170 feet wide by 78 feet high;
- Wastewater treatment retention basins – two one-acre basins;

- Stormwater retention pond – one 1-acre pond to collect runoff from buildings and parking areas; and
- Fuel storage – two 5,000 gallon storage tanks within containment pads, each 8 feet in diameter by approximately 13 feet in length.

These structures are expected to be supported on shallow spread and continuous footings or mat-type foundations.

A separate 21-acre satellite services complex near the south side of the site will contain maintenance and administration buildings (30,000 and 20,000 square feet, respectively), an additional stormwater retention pond, and two fuel storage tanks. Construction staging areas will be located immediately east of Hector Road.

The on-site substation will be located north of the railway line near the east side of the site. A 220 kiloVolt (kV) transmission line approximately 1.8 miles long will connect the on-site substation to the Pisgah Substation. Approximately 12 to 15 single circuit tower structures will be installed at a spacing of approximately 650 feet to 800 feet. The steel poles for the transmission line connection will be supported on CIDH piles.

Approximately 38 miles of paved roadways will be constructed for main travel routes, with approximately 250 miles of unpaved roads used between alternate rows of SunCatchers for construction and maintenance access. In addition, unpaved perimeter roads will be installed to provide security access along the perimeter fence lines. Polymeric stabilizers may be used in lieu of traditional road construction materials for paved roads or to stabilize unpaved roads. A bridge is proposed to cross the railway line near Hector Road. A temporary access road is proposed east of the site.

Earthwork will be kept to a minimum during site preparation, however, earthwork is required to establish grades for building sites, the substation, and paved arterial roads. Paved roadways will be constructed as close to the existing topography as possible, with limited cut and fill operations to maintain roadways at slopes less than 10 percent. Blading for unpaved roadways and foundations will occur between alternating rows of SunCatchers. Minor localized hills or depressions will be removed as needed to provide for proper alignment and operation. Minor cut and fill slopes will be constructed at 2:1 horizontal:vertical (H:V) or flatter. Culverts will be installed in a limited fashion as necessary for crossing of natural washes. In general, cuts and fills on the site will be localized.

A separate geotechnical study will be performed for an expansion to the Pisgah Substation and the proposed transmission line upgrades that are outside of the Project boundary.

1.2 SCOPE OF SERVICES

The scope of services for this preliminary evaluation included researching and reviewing previously published geologic maps, aerial photographs, and topographic maps of the site area, performing preliminary-level field reconnaissance and geologic mapping, and preparing this report.

The review of available information, and the results of the field reconnaissance and mapping, were used to develop preliminary conclusions regarding:

- General subsurface soil and groundwater conditions;
- Site seismicity;
- Seismic and geologic hazards including fault rupture, strong ground motion, liquefaction, lateral spreading, seismic settlement, landsliding, expansive or collapsible soil, and subsidence;
- Site coefficients and near-source factors in accordance with the 2007 California Building Code;
- Site grading considerations;
- Foundation installation and constructability considerations;
- Recommendations for further geotechnical and geologic investigations.

This study was preliminary in nature and did not include any intrusive subsurface explorations. Additional field investigations will be required to provide engineering data for final design and construction.

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SECTION 2 GEOTECHNICAL AND GEOLOGIC EVALUATIONS

Prior to beginning the field studies, published geologic information for the site and site-specific geotechnical data provided by SES were reviewed. The field studies included performing site reconnaissance and geologic mapping. No subsurface explorations or laboratory testing were performed.

2.1 PREVIOUS INVESTIGATIONS

A geotechnical investigation was performed on a small portion of the site north of the Pisgah Substation by C.H.J. Incorporated (2006). That geotechnical investigation included advancing four borings to depths of up to 46 feet below the ground surface, performing geotechnical laboratory testing, and performing field and laboratory testing for thermal and electrical resistivity. The report includes an evaluation of geologic hazards and recommendations for design and construction of the demonstration project, which has elements similar to those planned as part of the Solar One Project. That report is included in this report as Appendix A.

2.2 GEOLOGIC REVIEW AND FIELD STUDIES

Available published geologic information and geotechnical information were reviewed to develop an understanding of conditions on the subject site. Stereographic aerial photographs of the site were also analyzed to evaluate site conditions and fault hazards.

Field geologic mapping was performed during October 2008 within the proposed project site limits. Photo-based site plans with topographic data (25-foot contour intervals, interpolated by Stantec, the Project Engineer) were used for the field mapping. Field activities included general surficial mapping of the contacts between geologic units and measuring and recording structural data. Due to the size of the site and the preliminary scope of the evaluation, detailed mapping was only performed at selected locations that provided good geologic exposures. Appendix B presents additional details and photographs of key features observed during the field reconnaissance.

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SECTION 3 SITE CONDITIONS

Knowledge of the site conditions has been developed from a review of published information on the area's geology, a previous geotechnical investigation on a portion of the site, and the field program undertaken for the current study.

3.1 GEOLOGIC SETTING

3.1.1 Physiographic Setting

The Project site is located in the eastern-central portion of the Mojave Desert Geomorphic Province in an area known as Hector (Figure 1). The area is bounded to the north by the Cady Mountains, to the east by the Sleeping Beauty Mountains, Pisgah Crater to the south, and by Lake Manix and Troy Lake basins to the west. The area is primarily characterized by alluvial fans and washes that gently to moderately slope to the south from the foot of the Cady Mountains. A number of Oligocene- or Miocene-age basaltic and andesitic volcanic rock outcrops are located in the foothills of the Cady Mountains. Quaternary-age basalt flows from the Pisgah Crater bound the southern portion of the Project site. Lacustrine deposits from one of the high level fluctuations of Lake Manix overlap the southwestern portion of the site to elevations of approximately 1,825 feet Mean Sea Level (msl). Deposits from Lake Manix basin suggest lake fluctuations that began during the middle Pleistocene and continued though most of the Late Pleistocene (Jefferson 2003).

3.1.2 Regional Geology

The geology of the Mojave Desert region can be divided into two groups according to their inferred age: Pre-Cenozoic rocks (approximately 65 million years ago [mya] and older) and Cenozoic rocks (present to approximately 65 mya). The Pre-Cenozoic rocks represent the basement rocks of the present day Mojave desert region and are typically represented as mountains and rock outcrops. The Pre-Cenozoic rocks were subsequently overlain by Cenozoic rocks which are typically represented as volcanic mountains and flows, alluvial basins and valleys, and lacustrine lakebed deposits. Detailed descriptions of the two rock groups are provided below.

The Pre-Cenozoic rocks of the Mojave Desert region are generally made up of Pre-Cambrian (approximately 543 mya and older) gneiss and schist, and limestone, Mesozoic (approximately 65 mya through 245 mya) meta-volcanic rocks, and late Mesozoic granitic rocks, primarily composed of monzonites and granodiorites (Bassett and Kupfer 1964). All of these Pre-Cenozoic rocks underwent a period of regional metamorphism followed by period of deep erosion.

Throughout the Cenozoic, erosion of the Pre-Cenozoic rocks and more recent volcanic rocks resulted in the development of alluvial filled basins throughout the region. Lava flows from volcanic activity that occurred intermittently during the last 1.8 million years can also be seen in the region. Development of a series of lakes and their subsequent retreat happened primarily in the Pleistocene (0.01 to 1.8 mya) and resulted in lacustrine deposits stratigraphically above the existing Pre-Cenozoic and Cenozoic rocks (Diblee 1980a).

3.1.3 Local Geology

The geologic units in the Project vicinity are presented in the table below, Geologic Conditions, and are shown on Figures 3 and 5, Site Geologic Map and Field Reconnaissance Geologic Map.

Table 1
Geologic Conditions

Geologic Map Unit	Unit or Formation Name	Description/Comments
Qa	Quaternary alluvium	Late Pleistocene to Holocene; unconsolidated clay, silt, sand and gravel of alluvial fans and streamwash deposits, partly dissected and poorly sorted. Typically light reddish brown to light brown, Gravelly (~15%), fine to coarse Sand (~85% including eolian deposits), trace Cobbles. Granitic and volcanic clasts up to 8 inches, sub-angular to sub-round and moderately weathered.
Qf	Quaternary alluvial fan gravel	Late Pleistocene to Holocene; unconsolidated silt, sand, gravel and cobbles of slopewash, alluvial fans and streamwash deposits. Typically light reddish brown, Gravelly (~30%), coarse to fine Sand (~50%), with Cobbles (~20%). Granitic and volcanic clasts up to 18 inches, sub-angular to sub-round and moderately weathered.
Qb	Quaternary basalt of Pisgah flow	Holocene; dark gray Basalt, vesicular, moderately weathered and strong. Recent flows from nearby Pisgah Crater.
Qlc	Quaternary lacustrine deposits	Late Pleistocene to Holocene lake deposits; fine-grained dry lake bed deposits displaying mud cracks in localized surface depressions and in the low lying areas.
Qoa	Quaternary older alluvium	Pleistocene; moderately dissected, moderately consolidated, poorly sorted clay, silt, sand and gravel of older alluvial fans, terraces, and channel deposits. Typically light reddish brown to light brown, Gravelly (~15%), fine to coarse Sand (~85%), trace Cobbles. Granitic and volcanic clasts up to 8”, sub-angular to sub-round and moderately weathered.
Qof	Quaternary older fanglomerate and gravel	Pleistocene; partly dissected, largely unconsolidated silt, sand and gravel deposits of slopewash, older alluvial fans and terraces. Typically light reddish brown to light brown Sandy Gravel/Gravelly Sand with few Cobbles. Predominantly volcanic clasts up to 15 inches, sub-angular to sub-round and moderately weathered.
Tb	Oligocene or Miocene basalt	Oligocene or Miocene; inselberg forming volcanics, gray to dark gray, porphyritic, moderately vesicular, moderately weathered, strong.
Ta	Oligocene or Miocene andesite	Oligocene or Miocene; inselberg forming volcanics, light gray to gray, porphyritic, moderately weathered, strong.
Tab	Oligocene or Miocene andesitic breccia	Oligocene or Miocene; inselberg forming volcanics, light gray to gray, moderately weathered, strong. Flows and flow breccia composed predominantly of aphyric to porphyritic andesite.
Gqm	Mesozoic granite to quartz monzonite	Mesozoic; reddish brown to light brown, mountain forming, coarse-grained, subequigranular, moderately weathered, evident spheroidal weathering.

The Project site is near the toe of an alluvial fan emanating from the Cady Mountains located north-northeast of the Site. The alluvium is dissected by southwest-trending washes. Geologic mapping of the Site and surrounding areas show the Site is underlain by young alluvial fan deposits of Holocene (present to 0.01 mya) to late Pliestocene age. The alluvial deposits are overlain in part by Holocene basalt of the Pisgah flow (CHJ Incorporated 2006), and a number of Oligocene or Miocene basaltic and andesitic volcanic rock outcrops were mapped in the northeastern portion of the Project Site.

3.2 TECTONIC FRAMEWORK AND HISTORICAL SEISMICITY

The Mojave Desert Geomorphic Province is a wedge shaped area largely bounded by the San Andreas Fault Zone and the Garlock Fault and is structurally referred to as the Mojave Block. The Mojave Block is cut by a series of northwest to southeast striking faults as shown on Figure 4. Collectively, the strike slip faults in the Mojave Block are referred to as the Eastern California Shear Zone (ECSZ). The epicenters of historical earthquakes experienced in the area are also shown in Figure 4.

Significant faults within 62 miles (100 kilometers) of the center Project site are provided in Table 2 below. The faults are listed in order of proximity to of the Project site. Fault type, fault length, maximum estimated slip rate, and probable maximum earthquake magnitude are also listed in the table.

Table 2
Significant Faults Within 62 Miles (100 Kilometers)

Fault Name	Nearest Distance to Solar One Site	Type of Faulting[1]	Fault Length[1] miles (km)	Maximum Estimated Slip Rate inches/year (mm/year)[1]	Probable Maximum Earthquake Magnitude[1] (M_{max})
Lavic Lake	0	right-lateral strike-slip	17 (27)	Unknown	7.1
Pisgah	0	right-lateral strike-slip	21 (34)	0.04 (0.8)	6.0 – 7.0
Calico	14.0 (22.5)	right-lateral strike-slip	21 (95)	0.10 (2.6)	7.1
Camp Rock - Emerson	20.0 (32.2)	right-lateral strike-slip	22 (35)	0.04 (1.0)	6.8
Lenwood	28.0 (45.1)	right-lateral strike-slip	22 (35)	0.03 (0.8)	6.8
North Frontal Zone	35.0 (56.3)	thrust	40 (65)	0.04 (1.0)	7.1
Helendale	44.0 (70.1)	right-lateral strike-slip	31 (50)	0.03 (0.8)	7.3
Gravel Hills	45.0 (72.4)	right-lateral strike-slip	31 (50)	0.04 (0.9)	7.2
Pinto Mountain	46.0 (74.0)	left-lateral strike-slip	45 (30)	0.04 (1.0)	7.5

Table 2
Significant Faults Within 62 Miles (100 Kilometers)
(Continued)

Fault Name	Nearest Distance to Solar One Site	Type of Faulting[1]	Fault Length[1] miles (km)	Maximum Estimated Slip Rate inches/year (mm/year)[1]	Probable Maximum Earthquake Magnitude[1] (M_{max})
Garlock	53.0 (85.3)	left-lateral strike-slip	155.0 (250)	0.43 (11)	7.1
Death Valley	54.0 (86.9)	right-lateral strike-slip	71 (115)	0.12 (3.0)	7.3
San Andreas	56.0 (90.1)	right-lateral strike-slip	745 (1,200)	1.41 (36)	7.9
Cleghorn	58.0 (93.3)	left-lateral strike-slip	19 (30)	0.10 (3.0)	unknown

Notes:
[1] Data obtained from the Southern California Earthquake Center (SCEC) website. See References.

The following sections discuss significant faults in order of increasing distance.

3.2.1 Eastern California Shear Zone (ECSZ)

Geodetic studies have suggested that approximately 6 to 8 millimeters per year (mm/yr) of right-lateral slip are accommodated across the ECSZ (Sauber *et al.*, 1986). This movement represents approximately 15% of the motion between the Pacific and North American plates. Individual faults within the ECSZ have estimated slip rates of less than 1 mm/yr. These are relatively low slip rates when compared to the San Andreas fault (36 mm/yr) or the major faults west of the San Andreas in southern California that include the San Jacinto (12 mm/yr), Elsinore (6 mm/yr), Palos Verde (3 mm/yr) or the Newport-Inglewood faults (1.5 mm/yr). Given the relatively low slip rates of the faults in the ECSZ, the recurrence interval between moderate to large earthquakes on any of the these faults is relatively long, on the order of 5,000 years or longer.

Despite the long recurrence intervals estimated for moderate or large earthquakes on individual faults within the ECSZ, there have been two significant earthquakes in the region within the last 15 years. The 1992 Landers event ruptured along a series of faults in the central portion of the ECSZ, about 45 miles south of the project site. This Moment Magnitude (M_w) 7.3 event was accompanied by significant ground rupture, with over 18 feet of slip noted at certain locations, and over 3 feet of slip noted over 53 miles of the rupture.

In 1999, less than 7 years later, a M_w 7.1 event occurred on the Bullion and Lavic Lake faults (referred to as the Hector Mine earthquake). These events were located approximately 18 miles to the south of the project area. The overall length of ground rupture has been estimated at 28 miles with significant slip (greater than an inch or so) occurring over a distance of about 22 miles. Maximum displacement was estimated at 17 feet of right slip and an average slip of approximately 8 to 10 feet.

3.2.2 Pisgah-Bullion and Lavic Lake Fault Zones

Two faults within Earthquake Fault Zones (Alquist-Priolo Zones) are mapped on the project site as seen in Figure 2. The westernmost is the Pisgah fault and is considered part of the Pisgah-Bullion fault zone. The northern portion of the Bullion Fault is presumed to connect in the subsurface with the Pisgah Fault (Hart 1987). This fault zone is a right-lateral fault system and is considered by the State of California to have a M_w 7.1 as evidenced by the Hector Mine earthquake of 1999.

The second Earthquake Fault Zone projecting into the site is the northern extension of the Lavic Lake Fault Zone. It extends northwest from near the center of the length of the Pisgah-Bullion fault zone. It runs just east and parallel to the Pisgah-Bullion Fault Zone. Due to limited surface expression and young alluvial cover, the northernmost part of this fault zone was simply mapped as “Fault A and Fault B” (Hart 1987).

Shaking along these faults during the Hector Mine earthquake of 1999 is interpreted as producing as much as 510 mm horizontal motion near the epicenter. However, northward, towards the project site this displacement diminishes to as little as 2 mm of movement. No movement was recorded north of Interstate 40 or in the Project area during this event.

Recent observations of the Pisgah and Lavic Lake faults were made with stereo ortho-photographs and field reconnaissance mapping. Mapped interpretations of these fault projections can be seen in Figure 5.

3.2.3 Cady Fault and Unnamed Faults in the Cady Mountains

The Cady fault is an east-west trending fault that exists approximately 9 miles north of the project site in the northern flank of the Cady Mountains and runs for approximately 12 miles. It is a left-lateral, strike-slip fault. It is believed to have ruptured in the Quaternary and movement is shown in older alluvial deposits. However, younger alluvium overlays the eastern end of the fault which suggests no recent movement.

Two northeast trending faults that exist in the igneous rocks north and northeast of the project site are likely pre-Quaterary in age and recent faulting is not likely. The easternmost of these faults runs from the northeast corner of the Project site parallel to the existing transmission line to the northeast. The other fault runs northeast into the Cady Mountains from just north of the northwest corner of the Project site (Figure 3).

3.2.4 Calico Fault

The Calico Fault is a northwest-southeast trending right lateral, strike-slip fault that exists approximately 14 miles to the east of Project site (Figure 4). It has an estimated horizontal slip rate of 0.10 inches a year with a probable maximum M_w of 7.1. It is estimated to rupture every 1,500 years with the most recent rupture being March 18, 1997.

3.2.5 Camp Rock and Ludlow Faults

As is characteristic of major faults within the ECSZ, the Camp Rock and Ludlow Faults trend northwest-southeast and display right-lateral, strike-slip displacement. These faults are mapped to extend within 20 miles west and 12 miles east, respectively, of the Project site (Figure 4). The Camp Rock, Emerson, and Copper Mountain faults make up a roughly continuous fault system some 62 miles in length. About 12 miles of the Camp Rock fault ruptured in the Landers earthquake of 1992. The State of California (State) assigns a maximum magnitude earthquake of M_w 7.3 to the Camp Rock-Emerson Fault (Cao *et al.,* 2003). The State does not consider the Ludlow fault in recent hazard assessments.

3.2.6 Pinto Mountain Fault

The Pinto Mountain Fault forms the south-central boundary of the Mojave Desert block, truncating several of the northwest-trending faults characteristic to this region. The Pinto Mountain Fault is a left-lateral, strike-slip fault which has a significant vertical component of displacement (down-to-the-south) particularly in the western sections (Bryant 1986). This fault is located 46 miles south of the Project site and was assigned a M_w of 7.0 by the State seismic hazard assessment (Cao *et. al.,* 2003).

3.2.7 Garlock Fault Zone

The Garlock Fault zone marks the northern boundary of the Mojave Block and is one of the most obvious geologic features in southern California. It is a left-lateral strike-slip fault that connects at an acute angle to the San Andreas Fault Zone and trends northeasterly to its terminus in the northern Mojave Desert. The slip rate ranges from 2 to 11 millimeters per year, with a rupture interval ranging between 200 and 3,000 years. The Garlock Fault Zone is given a probable M_w of 7.6. The most recent earthquake with a M_w of 5.7 was on July 11, 1992 near the town of Mojave.

3.2.8 San Andreas Fault Zone

The San Andreas Fault extends northwest through California from the Salton Sea to Cape Mendicino, following a major zone of right-lateral crustal interaction between the Pacific and North American lithospheric plates. Mapped traces of the fault along the southwestern edge of the Mojave Desert block are located approximately 56 miles to the southwest of the project site. The State seismic hazard model (Cao *et. al.,* 2003) assigned a M_w 7.5 to the nearest portions of the San Andreas Fault.

3.3 SURFACE CONDITIONS

The Project site is generally part of coalescing alluvial fans emanating from the Cady Mountains located immediately north-northeast of the site. The alluvium is dissected by washes that trend southwest away from the base of the Cady Mountains. The depth and width of the washes is highly variable. The surface elevations slope gently to the southwest, from a high of about 2,600 feet msl on the north side of the site to about 1,800 feet msl at the southwest corner of the site. Greater topographic relief occurs in various rock outcrops in the northernmost part of the site.

The surface of the site is sparsely vegetated. Desert pavement (accumulation of gravel size material) has developed over much of the site. Hardly any vegetation occurs in the area of the lacustrine deposits in the southwest part of the site; the desert pavement is not well developed in those areas either.

3.4 SUBSURFACE CONDITIONS

The immediate Project site is chiefly underlain by light reddish brown to light brown Holocene- and Pleistocene-age alluvial deposits. To a lesser degree, lacustrine deposits were mapped along the southwestern portion. A number of Oligocene- to Miocene-age basaltic and andesitic volcanic outcrops were mapped in the northeastern portion of the Project site.

The subsurface conditions presented in this section are based on the geologic field mapping and a preliminary subsurface investigation (C.H.J. Incorporated 2006). The primary geologic units observed on the site are listed in Table 1. These units are described in detail in the following sections. The approximate geologic contacts between these units on the site, based on field mapping and review of aerial photographs, are shown in Figure 5, Field Reconnaissance Geologic Map.

3.4.1 Alluvium

Quaternary alluvium and fan gravel deposits dominate the site based on its location at the foot of the Cady Mountains located north-northeast of the site. The erosional debris shed from the mountains accumulates as coalesced alluvial fans and aprons at the toe of the mountain front. The fan gravels dominate the northeastern portion of the site, nearest the Cady Mountains, while alluvium covers the central and much of the southern portions of the site.

Older alluvial deposits occur as alluvium, fanglomerate and gravel, predominanatly in the southern-central portion of the Project site. The older alluvial deposits are generally dense to very dense and exhibit moderately mature desert pavement development when compared to Quaternary-age alluvium.

A preliminary geotechnical investigation was performed on a small portion of the site, north of the Pisgah Substation (C.H.J. Incorporated 2006). That investigation encountered loose near-surface deposits composed primarily of loose eolian dune sands on the order of 1 to 3 feet thick underlain by dense to very dense alluvial soils. The alluvial soils encountered consisted of poorly graded sand and silty sand, both with gravel. Drill rig refusal, attributed to nested cobble or boulder sized clasts, occurred in two of the borings at depths of 29 and 46 feet.

As part of the preliminary geologic reconnaissance and field mapping for the proposed project site, URS geologists performed two surface transect evaluations. The surface transect evaluations were designed and conducted to gain an understanding of the surface geology with respect to grain/clast size distribution. The surface transects were performed by following linear routes through relatively undisturbed segments of the geologic units found on the proposed project site in a down slope direction from the Cady Mountains.

Within the alluvial deposits (see table below) the percentage of sand generally increases with distance from the Cady Mountains, while the percentage of gravel and cobbles generally decreases. Further detail and photographs of the transects are presented in Appendix C, and transect locations are plotted on Figure 5.

Table 3
Transect Evaluations
Grain Size/Clast Size Distribution

Transect	Location	Sand (%)	Gravel (%)	Cobble (%)
1	1	40	40	20
1	2	40	40	20
1	3	60	30	10
1	4	70	25	5
1	5	75	20	5
1	6	80	15	5
1	7	95	5	0
1	8	95	5	0
1	9	95	5	0
2	1	80	20	0
2	2	90	10	0
2	3	90	10	0
2	4	80	20	0
2	5	95	5	0
2	6	95	5	0

3.4.2 Lacustrine Deposits

Lacustrine deposits were mapped along the southwestern portion of the Project site. The Pleistocene-age lacustrine deposits are from high-level fluctuations of Lake Manix as it overlapped the southwestern portion of the site. Deposits suggest lake fluctuations began during the middle Pleistocene and continued though most of the late Pleistocene (Jefferson 2003). In general, these Pleistocene dry lake bed deposits consist of interbedded fine-grained sand, silts and clays displaying mud cracks in localized surface depressions and in the low lying areas, with localized veneers of immature desert pavement.

3.4.3 Rock Outcrops

Numerous Tertiary-age basaltic, andesitic, and andesitic breccia volcanic rock outcrops were mapped in the northeastern portion of the Project site. These volcanic rocks are typically gray to dark gray, porphyritic, vesicular, moderately weathered and strong.

The Pisgah lava flows, which are mapped on the southwestern and southeastern edge of the Project site, originated from the Pisgah Crater and are quite extensive. Other flows are nearby, notably the Sunshine Peak and the Malpais flow of Newberry Mountains. They are believed to be late Quaternary-age and associated with the last activity of the Ludlow volcanic center. This area is believed to have been the main source area for the volcanic rocks in the southern and eastern Cady Mountains (Diblee 1980a).

3.4.4 Groundwater

A water well is present on the southern portion of Section 1 (T8N-R5E) as shown on Figure 5. The depth to groundwater was measured at 310 feet below the ground surface during a pumping test performed on the well during October 2008 (SES 2008).

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SECTION 4 SEISMIC AND GEOLOGIC HAZARDS

The primary geologic hazards at the Project site are surface rupture from one of the active faults on-site and strong ground motion from a seismic event centered on one of several nearby active faults. Evaluations of surface rupture, seismic shaking, liquefaction, expansive soil, subsidence and collapse, and slope stability at the site are discussed in detail below.

4.1 SURFACE RUPTURE

In 1972, the State of California passed the Alquist-Priolo Earthquake Fault Zoning Act to mitigate the hazard of surface faulting to structures for human occupancy. There are two mapped Earthquake Fault Zones that encroach upon the project site and can be seen on Figure 2. The western-most fault is the Pisgah Fault and the south-central one is the northern end of the Lavic Lake Fault. Surface expressions of these two faults were observed by URS geologists during a geologic reconnaissance and field mapping program performed from October 28 through October 31, 2008. A more detailed description of the field mapping program can be found in Appendix B. The traces of the faults observed during the field program are shown in Figure 5.

The potential for surface rupture of strands of the Pigah and Lavic Lake faults across the Project site is moderate. Additional evaluation of the fault strands will be performed during design-level geotechnical studies to confirm the presence and activity level of on-site faults. Recommendations for further evaluation of surface rupture are presented in Section 6.

4.2 STRONG GROUND MOTION

The site is within the Eastern California Shear Zone, an area of high seismicity and numerous active faults. Moderate to high levels of ground shaking could occur at the site as a result of an earthquake on any of a number of faults in the region, including the faults on site or the San Andreas, Imperial, Garlock, and Pinto Mountain faults. The Project is likely to be affected by an earthquake on one of these faults during the Project life.

4.3 LIQUEFACTION

Liquefaction is a process in which saturated soils lose strength because of earthquakes or other sources of ground shaking. The soil deposit temporarily behaves as a viscous fluid; pore pressures rise, and the strength of the deposit is greatly diminished. Liquefaction is often accompanied by sand boils, lateral spreading, and post-liquefaction settlement as the pore pressures dissipate. Liquefiable soils typically consist of saturated, cohesionless sands and silts that are loose to medium dense. Liquefaction is not typically thought to occur if groundwater is deeper than 50 feet below the ground surface.

The potential for liquefaction at the site was evaluated as part of the preliminary geologic and geotechnical evaluation for the Project. Loose granular materials may be present near the ground surface, however, groundwater is on the order of 300 feet below the ground surface. The depth to groundwater was measured at 310 feet below the ground surface during a pumping test performed on a well located on the southern portion of Section 1 (T8N-R5E) during October 2008 (SES 2008). Due to the depth to groundwater, the potential for liquefaction to occur at the site is low. Further, the Geologic Hazard Overlay in the San Bernardino County General Plan (URS 2007a) does not classify the site area as having a potential for liquefaction.

4.4 SECONDARY EFFECTS OF SEISMIC ACTIVITY

Secondary effects of seismic activity include seismically induced settlement of dry soils (seismic compaction), tsunamis, and seiches.

Seismically induced settlement of dry soils (seismic compaction) can occur during strong ground shaking in loose, clean granular deposits above the water table, resulting in ground surface settlement. The water table is on the order of 300 feet below the ground surface and granular soils exist above this level. Limited reconnaissance mapping and previous subsurface investigations (Appendix A) show that the granular soils at various locations onsite are denser that what would be considered loose deposits. Seismically induced settlement is not considered a significant hazard for most of the Project site.

The Project site is approximately 2,000 feet above sea level, and therefore the potential for flooding at the Project site as a result of a tsunami is considered to be very low. A wave created by earthquake shaking in an enclosed body of water is called a seiche. There are no significant bodies of water in the site vicinity. Therefore, the potential for flooding at the site as a result of a seiche is considered to be very low.

4.5 EXPANSIVE SOIL

Expansive soil and rock shrink and swell with changes in moisture content. Near-surface alluvial deposits on the Project site are expected to consist of primarily sand and gravel with a low expansion potential. Cohesive soil was not encountered in the borings advanced for the demonstration site (C.H.J. Inc. 2006). Some lacustrine soils were observed in the southwest portion of the site (see Figure 5). Visual observations indicated the soil has a high silt content, however, a potential exists for expansive material to be present. The likelihood for expansive soil to impact the project is judged to be low over the majority of the site and low to moderate in the southwest corner.

4.6 SUBSIDENCE AND COLLAPSE

The Mojave River area is subjected to subsidence from fluid withdrawal (generally associated with groundwater wells). Minor subsidence has been detected as close to the proposed project as the Troy Lake area to the west. The majority of the Project site is outside of the areas being monitored for subsidence within the Mojave River groundwater basin. The potential for damaging localized differential settlement from subsidence is considered low, given the measurements in the site vicinity and the limited groundwater lowering within the Project site. Further, the planned facilities are not highly sensitive to aerial settlement. While an increase in groundwater withdrawal is expected to occur as part of the Project, the impact to regional groundwater levels and subsidence is expected to be limited (Stamos, *et al.,* 2004; Sneed *et al.,* 2003).

Loosely deposited alluvium and colluvium can be subject to collapse due to wetting and/or inundation. The only areas of the site subject to significant saturation are within the washes. These areas have been inundated in the past, and are not likely to experience additional collapse settlement. Natural drainage patterns are not significantly changed as part of the project and the existing washes are excluded from development areas. Therefore, the project should not increase the potential for collapse settlement to occur at the site and the potential for collapse settlement to affect the project is low.

4.7 LANDSLIDES AND SLOPE STABILITY

Landslides can occur due to the presence of steep slopes, saturated soil or rock, and/or seismic activity. The majority of the site is on relatively level or gently sloping ground; therefore, the risk of land sliding is very low. The mountains on the northern site boundary have a low to moderate potential for landslide activity, based on preliminary observations. The Geologic Hazard Overlay in the San Bernardino County General Plan (URS 2007a) does not map the site within an area of landslide susceptibility. Based on the available information, the potential for landslides to affect the project is low.

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SECTION 5 GEOTECHNICAL CONSIDERATIONS

In our opinion, the site is geotechnically suitable for the proposed solar power plant. Below the loose sands encountered in the upper 1 to 3 feet, the underlying material is anticipated to be dense to very dense sand with gravel that should provide good support for deep foundations. The primary geotechnical and geologic considerations for design and construction include:

- The presence of loose sand within the upper 1 to 3 feet. Mitigation will likely be required to provide support for shallow foundations and other surface improvements.
- Installation of SunCatcher foundations through potentially dense and/or cobbly/bouldery soil.
- Strong seismic ground shaking and appropriate seismic design of project elements.
- Characterization of on-site faults and the avoidance of fault rupture hazard.

The following sections of this report present preliminary conclusions related to geotechnical design at the site. Preliminary 2007 California Building Code Seismic Coefficients are also presented. The potential for fault rupture was discussed in Section 4. The discussions and conclusions are based on literature research, results of current field studies, engineering evaluations, and professional judgment. The discussions are based on limited subsurface data and should be considered preliminary. Subsurface investigation will be required for final design.

5.1 EARTHWORK

Earthwork is likely to consist of minor grading for building foundations and pads and parking areas in the Main Services Complex and substation areas, as well as paved and unpaved roadways and utility trenches across the site.

Remedial grading will be required in portions of the site where structures and roads are constructed in areas of loose surficial soil. Near-surface soil encountered during the limited subsurface investigation for the demonstration site (C.H.J. Incorporated 2006) was found to be loose in some areas. Additional subsurface exploration will be performed to evaluate relative density, strength and compressibility across the broader site. At this preliminary stage, it is expected that overexcavation and recompaction of near surface soil will be required below foundations and roadways.

5.2 FOUNDATION CONSIDERATIONS

5.2.1 Shallow Foundations

Shallow foundations will likely be used to support light to moderate structures and equipment, primarily within the Main Services Complex and Satellite Services Complex. As discussed above, the near surface soil may be loose in some areas and is likely to require overexcavation and recompaction below shallow foundations. After recompaction, the soil at the site should provide moderate to high strength and low compressibility for the support of structures and equipment. Shallow strip and spread foundations are likely to be feasible for the majority of structures. Mat foundations may be required for larger structures or those sensitive to differential settlement.

5.2.2 Deep Foundations

Deep foundations that will likely be used for the SunCatchers and transmission line pole foundations should encounter moderate to high strength soil below the upper 1 to 3 feet. The deeper soils are expected to provide sufficient vertical and lateral support for these structures. The pipe-fin foundations planned for the SunCatchers are 24 inches in diameter and are vibrated into place.

Some of the borings for the demonstration site near the southeast corner of the project area encountered refusal at depths between 29 and 46 feet, likely due to the presence of cobbles and boulders. The particle size and frequency of cobbles and boulders is expected to increase closer to the Cady Mountains, to the north. If the pipe-fin foundations encounter refusal conditions on cobbles or boulders, or due to the presence of very dense sands, larger diameter drilled piers may be required. Drilled piers (also called cast-in-drilled hole [CIDH] piles) will also likely be the selected foundation type for the transmission line foundations. Large diameter (on the order of six feet) CIDH piles are likely to be able to penetrate areas where boulders are present. However, drilling fluid or casing may be required to reduce caving of the sides of the hole during drilling, and the extraction of boulders could increase the volume of concrete required to fill the holes. Subsurface investigations will be required to evaluate the presence of boulders and cobbles in the subsurface across the site.

5.3 SEISMIC DESIGN

Seismic design parameters developed from the 2007 California Building Code (CBC) are presented in this section. The table below provides 2007 CBC Seismic Coefficients for the central portion of the site near the proposed building locations.

Table 4
2007 CBC Seismic Coefficients

Parameter	Value	2007 CBC Reference
Site Class	D	Table 1613.5.2
Mapped Spectral Acceleration - Short Period, S_s (g)	1.168	Figure 1613.5[1]
Mapped Spectral Acceleration - 1 Sec. Period, S_1 (g)	0.389	Figure 1613.5[1]
Site Coefficient - Short Period, F_a	1.033	Table 1613.5.3(1)[1]
Site Coefficient - 1 Sec. Period, F_v	1.622	Table 1613.5.3(2)[1]
MCE [2] Spectral Response Acceleration - Short Period, S_{MS} (g)	1.207	Equation 16-37, $S_{MS}=F_aS_S$
MCE [2] Spectral Response Acceleration - 1 Sec. Period, S_{M1} (g)	0.631	Equation 16-38, $S_{M1}=F_vS_1$
Design Spectral Response Acceleration - Short Period, S_{DS} (g)	0.805	Equation 16-39, $S_{DS}=2/3*S_{MS}$
Design Spectral Response Acceleration - 1 Sec. Period, S_{D1} (g)	0.421	Equation 16-40, $S_{D1}=2/3*S_{M1}$

Notes:

1. Calculated using USGS program "Earthquake Ground Motion Parameters" Version 5.0.8 using site coordinates 34.80305 North, -116.40416 West.
2. MCE – Maximum Considered Earthquake.

The selection of Site Class D will require further evaluation after completion of the future geotechnical subsurface explorations.

5.4 RETENTION AND EVAPORATION BASINS

We understand that provisions will be made for providing evaporation and retention basins within the Main Services Complex. Infiltration rates were not measured as part of the limited subsurface investigation. It is our experience that permeability of on-site materials should range from moderate to high permeability for the onsite alluvium. Recommendations for permeability for the materials at the bottom of the basins should be further evaluated once the design plans are finalized. Field tests, such as infiltration tests, should be considered for inclusion in the final geotechnical investigation to measure infiltration rates.

5.5 PAVEMENTS

We understand that paved roadways will be constructed for main travel routes, with unpaved roads used between alternate rows of SunCatchers for construction and maintenance access. In addition, unpaved perimeter roads will be constructed to provide security access along the perimeter fence lines. Paved roadways will be constructed as close to the existing topography as possible, with limited cut and fill operations. Blading for unpaved roadways and foundations will occur between alternating rows of SunCatchers.

Polymeric stabilizers may be used in lieu of traditional road construction materials for paved roads or to stabilize unpaved roads. However, the property enhancements to the subgrade by polymeric stabilization are not known at this time. Further field studies and analyses will be required to provide pavement structural sections. Recommendations for both asphalt-paved roads and stabilized unpaved roads will be provided in the final geotechnical report.

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SECTION 6 ADDITIONAL GEOTECHNICAL SERVICES

Subsurface investigation will be required to provide geotechnical information for engineering design. Additional geologic review of fault hazards for the Pisgah fault and Lavic Lakes fault, including fault trenching, will also be performed. The following field activities are recommended:

- Hollow stem auger borings;
- Test pits excavated by backhoe;
- Evaluation of shear wave velocity in the upper 30 meters to support a Probabilistic Seismic Hazard Assessment (PSHA);
- Fault trenching across suspected active fault traces within the site;
- Geotechnical laboratory testing;
- Field and laboratory electrical and thermal resistivity testing; and
- Field permeability/infiltration testing.

A combination of borings and test pits is expected to be required. Borings will provide data on soil strength and compressibility, however refusal is likely to be encountered on cobbles and/or boulders in some locations, especially in the northern portion of the site. It is noted that little recovery was obtained in undisturbed Modified Califormia samplers, and the majority of soil samples were obtained using Standard Penetration Test (SPT) samples. Test pits will provide a visual interpretation of the distribution of particle sizes, particularly where a significant percentage of cobbles and boulders are present.

Following the field investigation, this report will be revised to include the field and laboratory data, as well as the results of additional engineering evaluations and analyses. To provide an estimate of the ground motions expected at the site, a PSHA will also likely be required. The probabilistic analysis incorporates the contribution of all known active faults near the site for which published data are available. The analysis attempts to account for uncertainty in rupture size, rupture location, magnitude, and frequency, as well as uncertainty in the attenuation relationship.

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SECTION 7 UNCERTAINTY AND LIMITATIONS

The discussions and conclusions presented in this report are based on limited research and non-intrusive field observations. Subsurface investigation will be required to obtain data for use in performing engineering analyses in support of final project design. Depending on the results of future studies, the conclusions presented herein may require revision.

Geotechnical engineering and the geologic sciences are characterized by uncertainty. Professional judgments presented herein are based partly on our understanding of the proposed construction, and partly on our general experience. Our engineering work and judgments rendered meet current professional standards; we do not guarantee the performance of the project in any respect.

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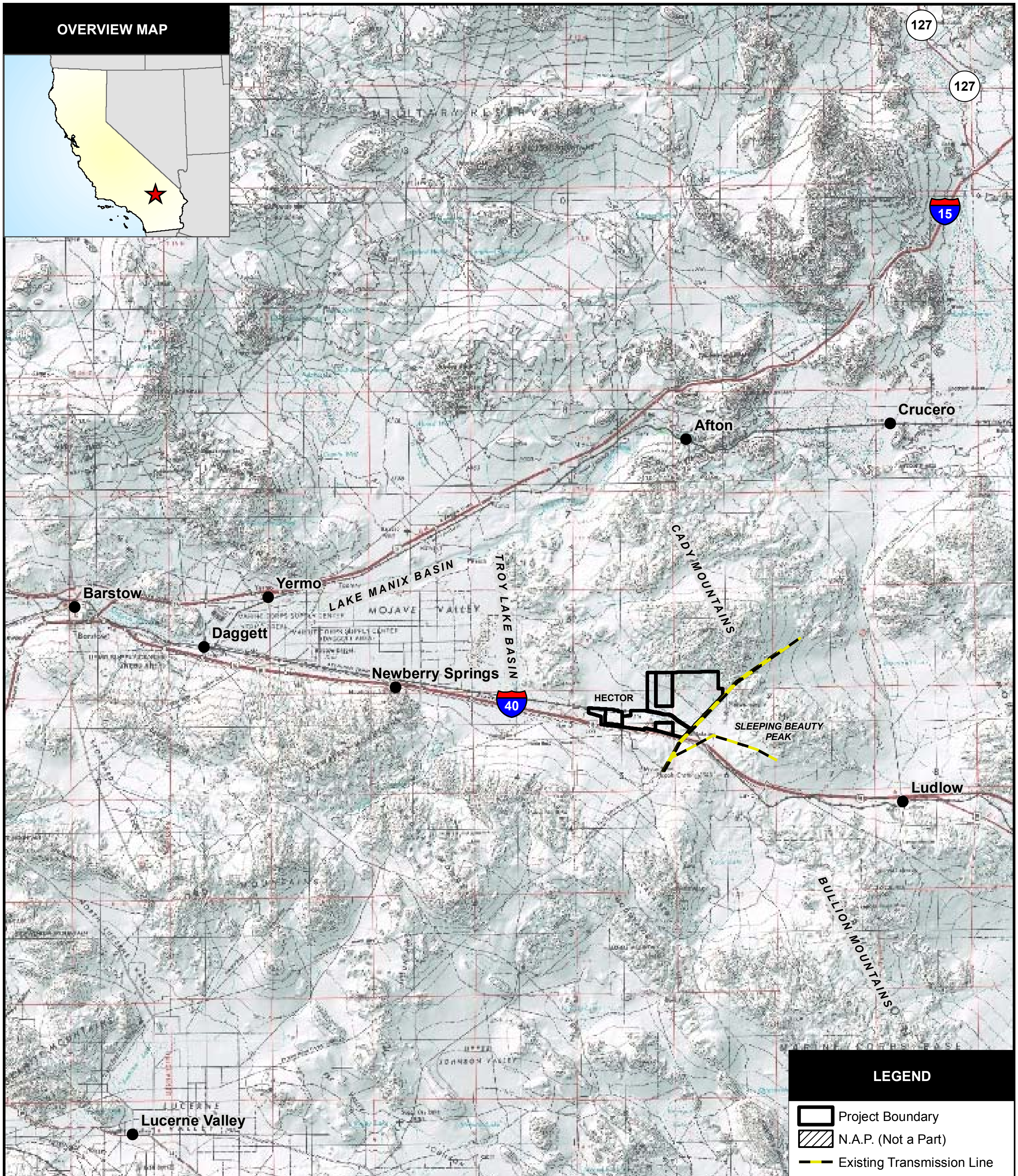
Figures



SOURCES: ESRI (overview);
Stantec Engineering (project site Oct. 2008);
USGS (7.5' quads various dates).

REGIONAL VICINITY MAP
SOLAR ONE PROJECT

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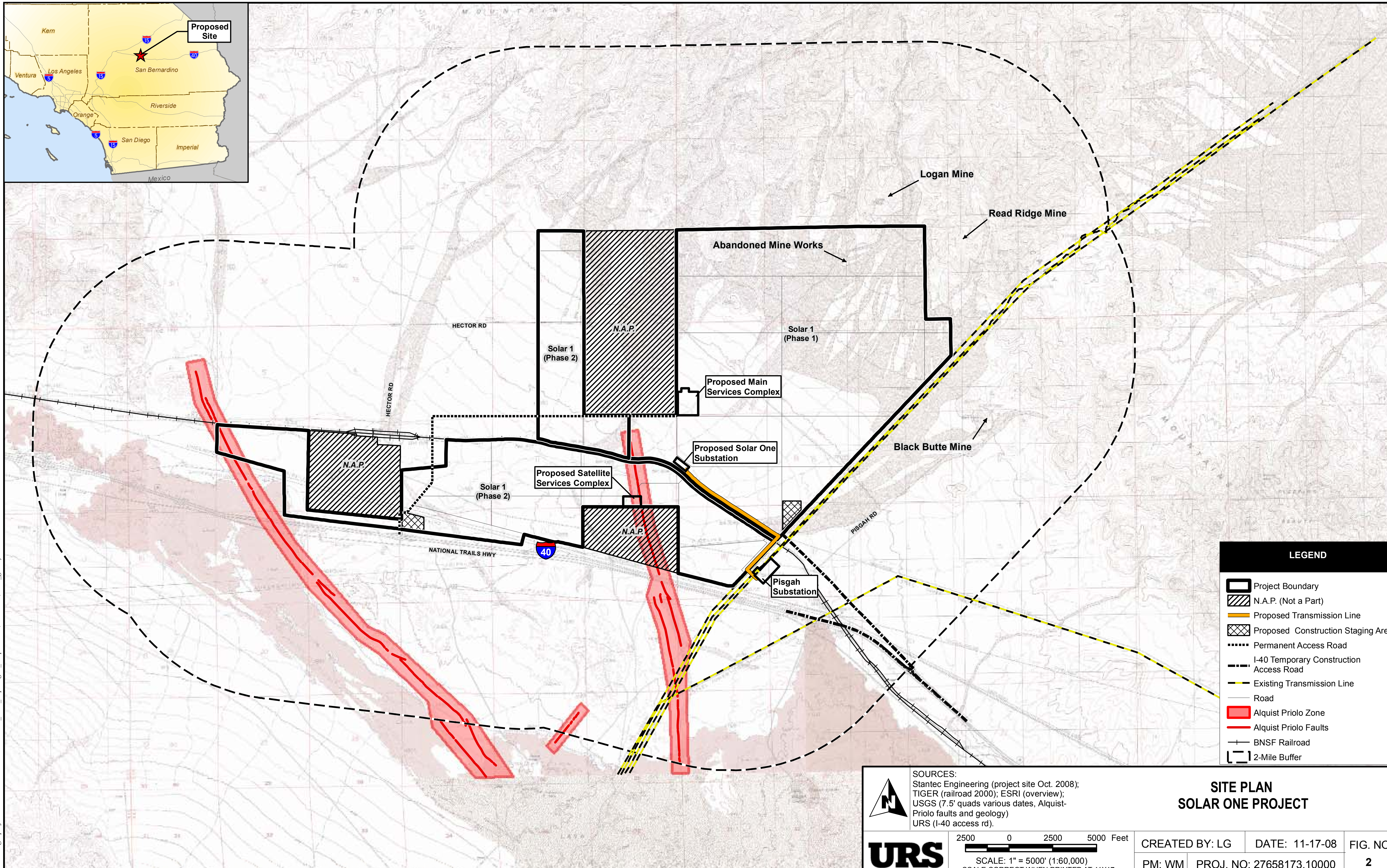

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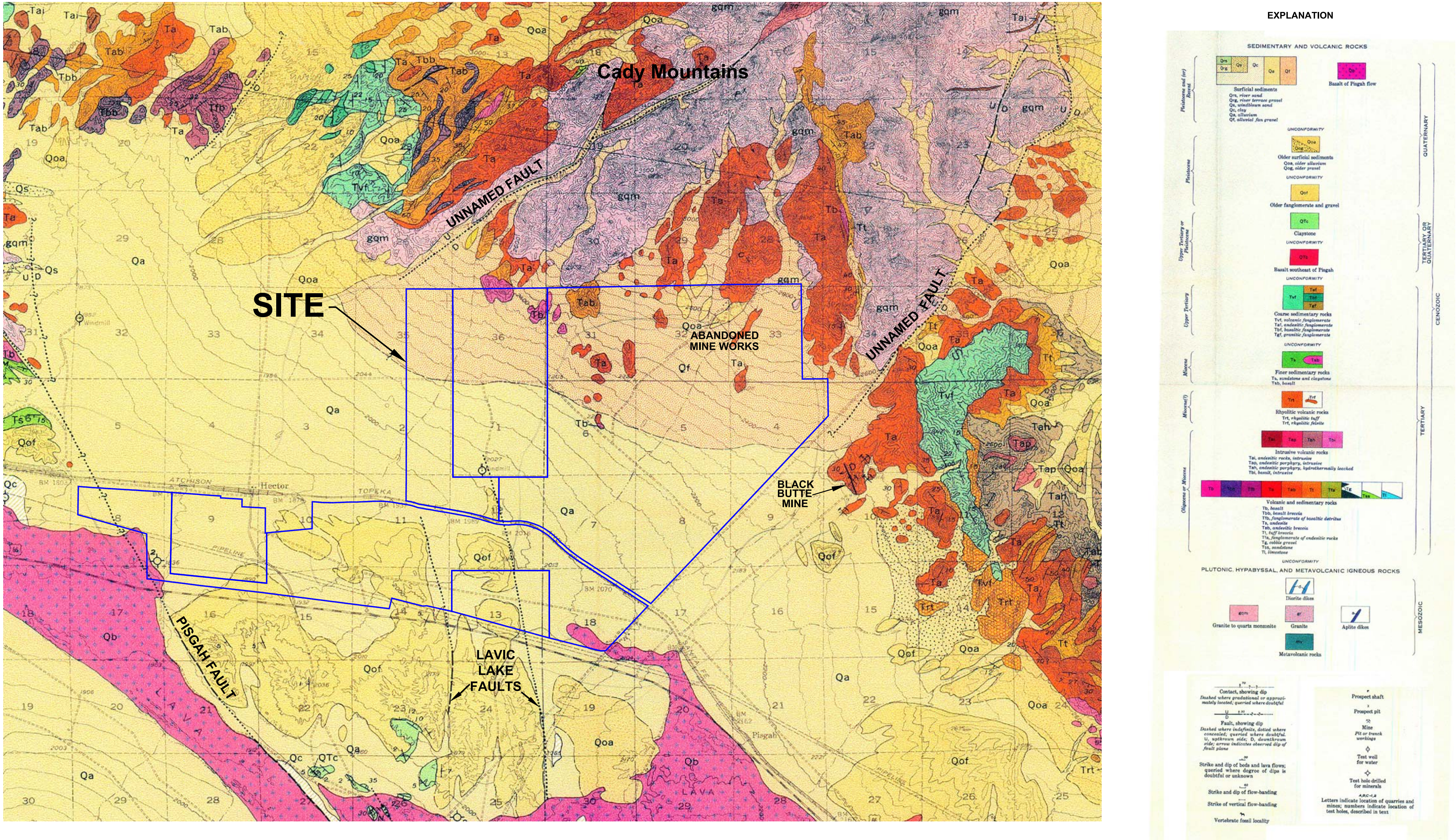

SOURCE:
Geologic Map of the Cady Mountains, San Bernardino County, California, by T.W. Dibbee, Jr., and A. M. Bassett

SITE GEOLOGIC MAP
SOLAR ONE PROJECT

URS

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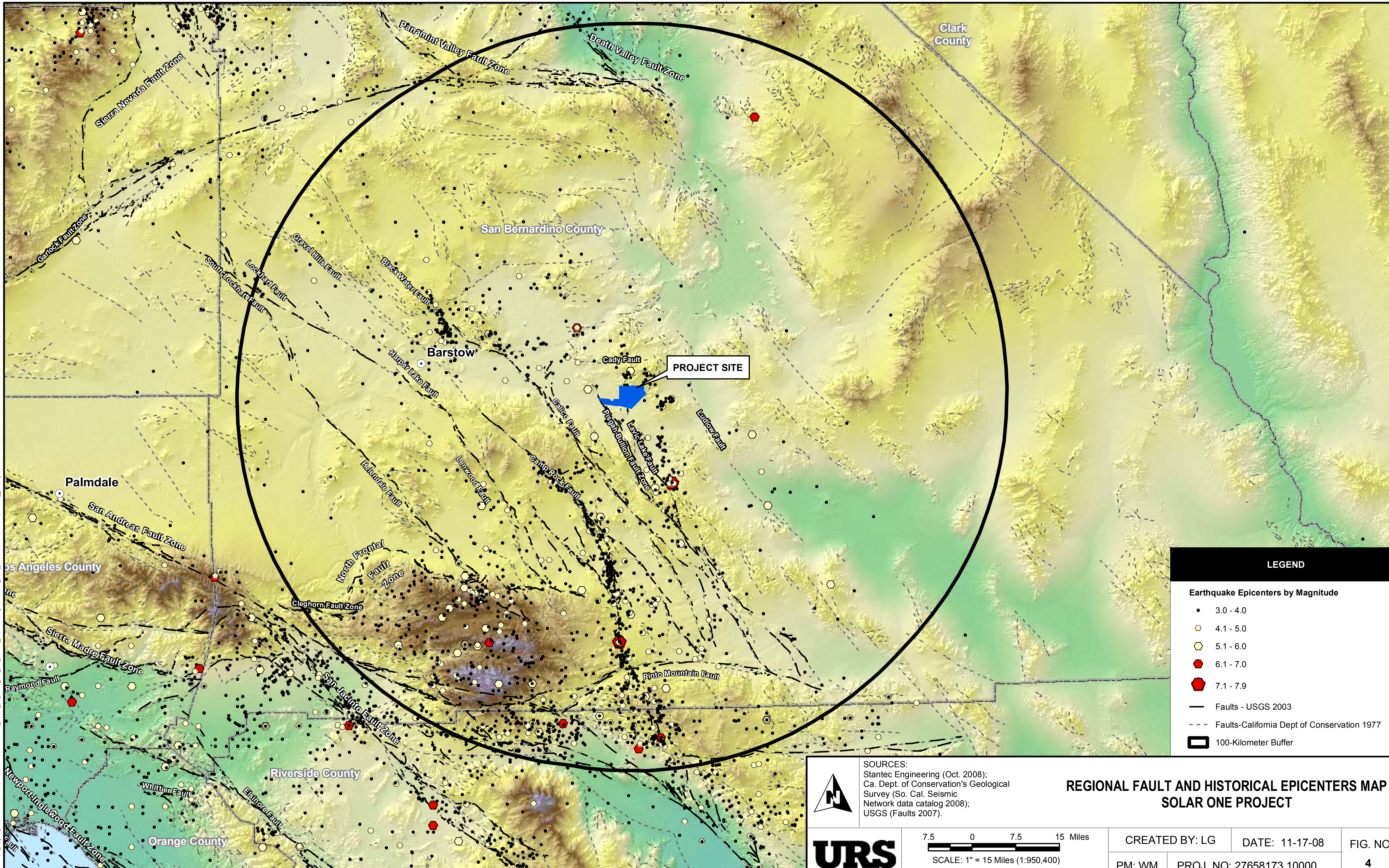

SOURCES:
Stantec Engineering (Oct. 2008);
Ca. Dept. of Conservation's Geological Survey (So. Cal. Seismic Network data catalog 2008);
USGS (Faults 2007).

REGIONAL FAULT AND HISTORICAL EPICENTERS MAP
SOLAR ONE PROJECT

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SCALE CORRECT WHEN PRINTED AT 11X17

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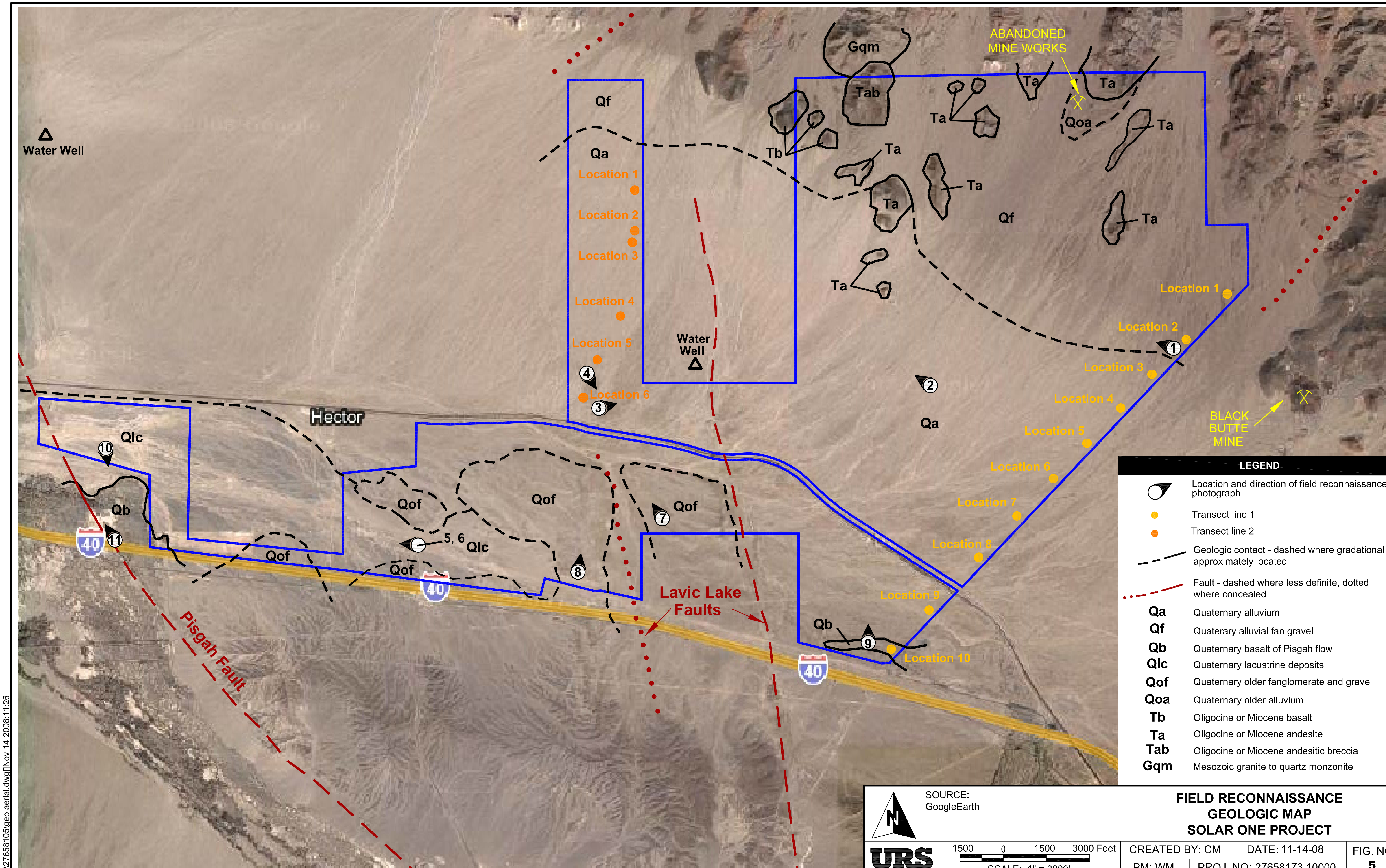

SOURCE:
GoogleEarth

FIELD RECONNAISSANCE
GEOLOGIC MAP
SOLAR ONE PROJECT

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**GEOTECHNICAL INVESTIGATION
PROPOSED SOLAR ONE-DEMONSTRATION SITE
NEWBERRY SPRINGS/LUDLOW AREA
SAN BERNARDINO COUNTY, CALIFORNIA
PREPARED FOR
STIRLING ENERGY SYSTEMS, INC.
JOB NO. V06100-3**

C.H.J. Incorporated
1355 E. Cooley Drive, Colton, CA 92324 ◆ Phone (909) 824-7210 ◆ Fax (909) 824-7209
15345 Anacapa Road, Suite D, Victorville, CA 92392 ◆ Phone (760) 243-0506 ◆ Fax (760) 243-1225

October 31, 2006

Stirling Energy Systems, Inc.
2920 East Camelback, Suite 150
Phoenix, Arizona 85016
Attention: Ms. Erika Hanson

Job No. V06100-3

Dear Ms. Hanson:

Attached herewith is the Geotechnical Investigation report prepared for Stirling Energy Systems' proposed Solar One - Demonstration Site, Pisgah Crater Road, San Bernardino County, California.

This report was based upon a scope of services generally outlined in our proposal letter, dated July 13, 2006, and other written and verbal communications.

We appreciate this opportunity to provide geotechnical services for this project. If you have questions or comments concerning this report, please contact this firm at your convenience.

Respectfully submitted,
C.H.J., INCORPORATED

Ben Williams

Ben Williams, P.G.
Senior Staff Geologist

BW/ADE:sra

Distribution: Stirling Energy System, Inc. (4)
Stantec, Inc. (2)

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NEWBERRY SPRINGS/LUDLOW AREA
SAN BERNARDINO COUNTY, CALIFORNIA
PREPARED FOR
STIRLING ENERGY SYSTEMS, INC.
JOB NO. V06100-3

INTRODUCTION

During September and October of 2006, this firm performed a geotechnical investigation for the proposed Solar One-Demonstration Site located west of Pisgah Crater Road and north of Interstate 40 in the Newberry Springs/Ludlow area of San Bernardino County, California. The purpose of the investigation was to explore and evaluate the geotechnical and soil corrosivity conditions at the subject site and provide appropriate geotechnical recommendations for design and construction of the proposed project. It is anticipated that buildings will utilize conventional shallow spread foundations, and caisson foundations will be utilized for support of the solar mirrors.

To orient our investigation, an undated aerial photograph (Map) showing the site and surrounding area was furnished for our use by Stantec, Inc. The Map indicated the desired locations of proposed borings, as well as thermal and electrical resistivity soundings. The locations of proposed structures, other improvements, and existing and proposed elevations were not indicated on the Map (Enclosure "A-1").

The results of our investigation, together with our conclusions and recommendations, are presented in this report.

SCOPE OF SERVICES

The scope of services provided during this geotechnical investigation included the following:

- Review of published and unpublished literature and maps
- Placement of four exploratory borings
- Logging and sampling of the exploratory borings for testing and evaluation
- Placement of six trenches for thermal resistivity testing by our consultant
- Testing by our subconsultant for electrical resistivity at nine locations
- Laboratory testing on selected samples

- Engineering geologic evaluation of geologic hazards
- Evaluation of the geotechnical data to develop site-specific recommendations for site preparation and grading, foundation design for both conventional spread foundations and caisson foundations, as well as mitigation of potential geotechnical constraints

PROJECT CONSIDERATIONS

It is our understanding that the Solar One-Demonstration Site is to be developed with a maintenance/administration building, a collector substation, and an array of solar mirrors. We further understand that the building and substation are to utilize conventional spread foundations for support, with the mirrors being supported by cast-in-drilled-hole (CIDH) caissons 6 to 8 feet in diameter and embedded approximately 18 feet in depth.

Neither the project grading plan nor foundation plans were available at the time of our investigation. It is our understanding that the elevations of the proposed improvements will be near the existing elevations. As such, significant cuts, fills, or significant slopes are not anticipated. The final grading plan should be reviewed by the geotechnical engineer.

SITE DESCRIPTION

The subject site is located southwest of Pisgah Crater Road and the Burlington Northern Santa Fe (BNSF) railroad tracks in the Newberry Springs/Ludlow area of San Bernardino County. Interstate 40 is located approximately 1/2 mile south of the site (see Enclosure "A-1"). The approximate latitude and longitude of the center of the site are 34.77° north and 116.35° west, respectively.

The site is crossed by two east-west trending buried gas transmission mains. Overhead electrical transmission lines and steel towers exist immediately east of the site. An electrical substation (Pisgah Substation) is located southeast of the site, along Pisgah Crater Road. Graded dirt maintenance roads associated with the railroad and the northern gas pipeline traverse the north portion of the site from east to west. Other than Interstate 40, the Burlington Northern Santa Fe railroad, the transmission lines, and substation, the area is generally an open desert. No structures were observed on the site.

Site topography consists of a roughly planar windblown dune sand surface with areas of gravel underlain by sand. Wash and sand dune areas present localized relief on the order of 4 to 5 feet across the site. In general, drainage on the site consists primarily of sheet flow. Two natural drainages were observed to traverse the site. Localized slopes up to 5 feet in height with a maximum inclination of approximately 2 horizontal to 1 vertical [2(h):1(v)] occur in the areas of the northern drainage channel. Evidence of recent flooding was not observed on the site. Vegetation consists of a sparse growth of desert grasses and shrubs, including creosote bushes. No other surface features pertinent to this investigation were noted during the site reconnaissance.

Limited aerial photograph coverage of the site and vicinity was reviewed, dating back to November 1952. The aerial photographs reviewed showed the site as vacant land. Adjacent properties appear to be undeveloped vacant land in the aerial photographs reviewed. No evidence of recent flooding was visible in the aerial photographs reviewed (or at the site), but it is possible that areas of the northwestern portion of the site may be flooded during thunderstorm activity. When the railway was constructed, the natural drainage system was apparently changed into a single drainage channel traversing the northwest portion of the site.

FIELD INVESTIGATION

The soil conditions underlying the subject site were explored by means of four exploratory borings drilled to a maximum depth of 46 feet below the existing ground surface (bgs). The borings were drilled utilizing a four-wheel drive truck-mounted CME 75 drill rig equipped with an automatic hammer for soil sampling. The approximate locations of our exploratory borings are indicated on the attached Plat (Enclosure "A-2"). Due to the granular non-cohesive sand deposits blanketing the site, it was necessary to utilize mats placed beneath the wheels of the drill rig to access the boring locations.

Continuous logs of the subsurface conditions, as encountered within the exploratory borings, were recorded at the time of drilling by a staff geologist from this firm. An attempt was made (Exploratory Boring No. 2) to obtain relatively undisturbed samples by driving a split-spoon ring sampler (California sampler) ahead of the borings at selected levels. Due to the high relative density of the soils, the granular non-cohesive nature of the soils, and the gravel/cobble content of the soils, obtaining such samples was not possible. As such, a standard penetration test (SPT) sampler was utilized in the remaining borings to obtain samples for classification purposes.

After the required seating of the sampler, the number of hammer blows required to advance the sampler a total of 12 inches was converted to equivalent standard penetration test (SPT) data and recorded on the boring logs. The number presented on the boring logs is the equivalent SPT-N value, which has been corrected for sampler size (California sampler vs. SPT sampler) and hammer efficiency. Bulk samples of typical soil types obtained were returned to the laboratory in sealed containers for testing and evaluation.

LABORATORY INVESTIGATION

Included in our laboratory testing program were field moisture content tests on all samples returned to the laboratory. The results are included on the boring logs. Sieve analyses were performed on selected soils as an aid to classification. Optimum moisture content - maximum dry density relationships were established for typical soil types. Direct shear tests were performed on selected remolded samples in order to provide shear strength parameters for bearing capacity and earth pressure evaluations. Selected samples of material were delivered to our subconsultant, Schiff Associates, for soil corrosivity testing.

The laboratory test results are presented in Appendix "C".

SOIL THERMAL AND ELECTRICAL RESISTIVITY MEASUREMENTS

Our subconsultant, Schiff Associates, performed soil thermal resistivity measurements at six locations and electrical resistivity measurements at nine locations. The locations of the measurements are indicated on the attached Plat (Enclosure "A-2"). The results of the tests and a discussion of the results will be forwarded under separate cover when received.

SITE GEOLOGY AND SUBSURFACE SOIL CONDITIONS

The site is located within the Mojave Desert geomorphic province that includes the northwest-trending faults of the Eastern California Shear Zone (ECSZ). The Mojave Desert geomorphic province is bounded on the southwest by the San Andreas fault and the Transverse Ranges (locally San Bernardino Mountains) and on the northeast by the Garlock fault. The region is characterized by fault block mountains and basins (horsts and grabens), possibly the result of mid to late Tertiary regional extension in response to the inception of movement on the San Andreas and Garlock faults. Erosional debris shed from the mountains generally accumulates as alluvial fans and aprons at the toe of the mountain fronts.

The alluvial plains and playas infilling the basins are generally formed of coalesced alluvial fans emanating from the adjacent mountains and intermittent lake deposits. In addition to the alluvial fans, extensive erosion and range front retreat have produced bedrock pediments in topographic continuity with the adjacent alluvial fans and plains around the base of many of the mountain ranges.

The site is located near the toe of an alluvial fan emanating from the Cady Mountains located north-northeast of the site. Regional geologic mapping of the site and surrounding areas show the site underlain by young alluvial fan deposits of Holocene to late Pleistocene age (Morton and others, 1980; Hart, 1987). The alluvial deposits are overlain in part by Holocene to early Pleistocene basalt of the Pisgah flow further to the south and west of the site (Morton and others, 1980; Hart, 1987). South of Interstate 40, older alluvial deposits overlain in part by late Pleistocene basalt of the Sunshine Lava Field were mapped by Morton and others (1980) and Hart (1987). Though not identified on published geologic maps of the site, fill materials up to several feet in depth are anticipated to exist along the east-west trending buried transmission gas main alignments. Minor surficial fill deposits are also anticipated to exist along the graded access road in the northern portion of the site. A Geologic Index Map is included as Enclosure "A-3".

As encountered within our exploratory borings and trenches, the upper 1 to 3 feet of native soils consist of wind blown dune sands. These non-cohesive soils are in a loose state, having been disturbed by plant growth and burrowing animals. The loose sand deposits are underlain at relatively shallow depths by dense to very dense alluvial soils, consisting of poorly graded sand and silty sand, both with gravel. Localized gravelly lenses were encountered within the exploratory borings to the maximum depths attained. Although not returned in the samplers or drill cuttings, the drill rig encountered cobble- to small boulder-sized clasts in each boring.

The soils encountered were sufficiently granular to preclude a potential for significant expansion.

Neither groundwater nor bedrock was encountered within any of the exploratory borings or trenches to the maximum depths attained. See the <u>GROUNDWATER AND LIQUEFACTION</u> section of this report for further discussion of groundwater.

Refusal to further advancement of the augers was experienced within Exploratory Boring Nos. 1 and 3 at depths of 29 and 46 feet bgs, respectively. Based on our experience, it appears that the refusal was due to nested cobble- or boulder-sized clasts.

All of our exploratory borings experienced significant caving upon removal of the augers.

A more detailed description of the subsurface soil conditions encountered within our exploratory borings is presented on the attached boring logs (Appendix "B").

The results of the soil corrosivity testing are discussed within the section titled SOIL CORROSION.

FAULTING

The tectonics of the Southern California region are dominated by the interaction of the North American Plate and the Pacific Plate, which are sliding past each other in a transform motion. Although some of the motion may be accommodated by rotation of crustal blocks such as the western Transverse Ranges (Dickinson, 1996), the San Andreas fault zone is thought to represent the major surface expression of the tectonic boundary and to be accommodating most of the lateral motion between the Pacific Plate and the North American Plate. However, some of the plate motion is accommodated along other northwest-trending strike-slip faults that are thought to be related to the San Andreas system, such as the San Jacinto fault and faults associated with the ECSZ. Local compressional or extensional strain resulting from the lateral motion along this boundary is accommodated by left-lateral, reverse, and normal faults such as the Pinto Mountain fault and the North Frontal fault zone (Matti and others, 1992; Morton and Matti, 1993).

EASTERN CALIFORNIA SHEAR ZONE:
The site is located within the eastern portion of the ECSZ, a zone of distributed dextral shear that includes a system of predominantly northwest-trending, strike-slip faults traversing the Mojave Desert. The ECSZ accommodates strain between the Pacific/North American Plate boundary across a zone approximately 105 kilometers (65 miles) wide and is thought to transfer as much as 15 percent of the total plate boundary shear into the Great Basin area (Shermer and others, 1996). A number of faults of this system, including the Camp Rock-Emerson fault, located 30 kilometers southwest of the site, ruptured in combination during the 1992 Landers earthquake. The more recent Hector Mine earthquake of 1999 occurred on a fault within the ECSZ known as the Lavic Lake fault located approximately 8 kilometers south of the site and included rupture along the central portion of the Bullion fault.

The Pisgah and Bullion faults are considered major components of the ECSZ and are located 5 kilometers west-southwest of the site and 13 kilometers south of the site, respectively. The Bullion fault is considered to be a southeastward continuation of the Pisgah fault (Morton and others, 1980; Hart, 1987). Rupture along these faults may occur in series as a single system. The Bullion fault is capable of rupturing in conjunction with the nearby Lavic Lake fault as evidenced by the Hector Mine earthquake. Cao and others (2003) have assigned a slip rate of 0.6 millimeters per year to the Pisgah-Bullion system.

The Ludlow fault, mapped as potentially active by Jennings (1994), is located approximately 16 1/2 kilometers east-northeast of the site and is included within the ECSZ. While ground-based data do not support classification of the Ludlow fault as active according to State guidelines, satellite interferometry measurements collected prior to and four days after the Hector Mine earthquake suggest that the Ludlow fault forms the eastern boundary of the ECSZ as the easternmost active fault structure (Sandwell, et al., 2000).

The site is not located within or immediately adjacent (within 1/4 mile) to an Alquist-Priolo Earthquake Fault Zone (APZ), designated by the State of California to include traces of suspected active faulting. An unnamed fault, most likely an extension of the Pisgah fault, is located approximately 0.8 kilometer west of the site (Morton and others, 1980; Hart, 1987). This fault, as well as the Pisgah and Bullion faults, are considered to be Holocene active; as such, they are included within an APZ (Enclosure "A-5"). Evidence of active faulting on or immediately adjacent to the site was not observed during the geologic field reconnaissance or on the aerial photographs reviewed.

NORTH FRONTAL FAULT ZONE:

The North Frontal fault zone forms the boundary between the Mojave Desert geomorphic province and the Transverse Ranges geomorphic province to the south and accommodates uplift of the northern San Bernardino Mountains. This complex zone of left-lateral, thrust, and reverse faults and folds is coincident with the northern boundary of the San Bernardino Mountains and is associated with convergent tectonic deformation that forms the regional relief. Fold structures exhibit decreasing age of formation with increasing distance from the mountain front based on geomorphic indicators (Eppes et al., 2002). The Ord Mountains fault, located approximately 50 kilometers southwest of the site, is a north-northeast trending zone of low angle reverse faults and high angle faults (Bryant, 1986) including the Apple Valley Highlands, Deep Creek, Juniper Ranch, and Powerline Road faults. Meisling (1984) has assigned a preferred late Quaternary slip rate of 0.14 mm/yr to the Apple Valley Highlands and Deep Creek faults. The Juniper Ranch and Powerline Road faults are considered to be inactive (Meisling, 1984). Portions of the North Frontal fault are included within an APZ.

SAN ANDREAS FAULT ZONE:
The postulated structural boundary between the San Bernardino and Mojave segments of the San Andreas fault zone is located approximately 115 kilometers southwest of the site. The toe of the mountain front in the San Bernardino valley area roughly demarcates the presently active trace of the San Bernardino mountains segment. The northeastern boundary of the western Transverse Ranges demarcates the trace of the Mojave segment.

HISTORICAL EARTHQUAKES

A map of recorded earthquake epicenters is included as Enclosure "A-4" (Epi Software, 2000). This map includes the Cal Tech database for earthquakes with **M** 4.0 or greater from 1977 through 2006.

In April and June of 1992, the Southern California region experienced shaking from the Joshua Tree and Landers earthquakes of **M** 6.1 and 7.3, respectively. The Joshua Tree and Landers events were located within a complex system of north to northwest-trending faults, including the Johnson Valley, Homestead Valley, and Emerson-Camp Rock faults of the ECSZ. A distinct aftershock pattern from these earthquakes occurred in the Barstow area. A **M** 6.4 earthquake occurred in Big Bear three hours after the 1992 Landers earthquake but has not been attributed to a specific fault.

On October 16, 1999, another large earthquake occurred along faults of the ECSZ. This **M** 7.1 earthquake was named the Hector Mine earthquake after a nearby feature. This earthquake was generated by slip along the Lavic Lake and Bullion faults. The relatively short time frame between the Landers and Hector Mine earthquakes has led some investigators to suspect that the Mojave Desert is currently in a cycle of high seismicity.

No large historical earthquakes have occurred on the San Bernardino Mountains segment of the San Andreas fault (Working Group on California Earthquake Probabilities, 1988), although Jacoby and others (1987) suggest one of two major earthquakes in 1812 may have occurred on this segment. Surface rupture occurred on the Mojave segment of the San Andreas fault in the great **M** 8+ 1857 Fort Tejon earthquake. The Coachella Valley segment of the San Andreas fault was responsible for the 1948 Richter magnitude 6.5 earthquake in the Desert Hot Springs area and for the 1986 Richter magnitude 5.6 earthquake in the North Palm Springs area. The Working Group on California Earthquake Probabilities (1995) tentatively assigned a 28 percent (±13 percent) probability to a major earthquake occurring on the San Bernardino Mountains segment of the San Andreas fault between 1994 and 2024. The corresponding probability for the Mojave segment is 26 percent (±11 percent).

The North Frontal fault zone has no historical record of large earthquakes but is seismically active, with many M_L 4+ earthquakes having been recorded in the complex area of the intersection with the Helendale fault of the ECSZ (Bryant, 1986).

SEISMIC ANALYSIS

The precise relationship between magnitude and recurrence interval of large earthquakes for a given fault is not typically known due to the relatively short time span of recorded seismic activity. As a result, a number of assumptions must be made to quantify the ground shaking hazard at a particular site. Seismic hazard evaluations can be conducted from both a probabilistic and a deterministic standpoint. The primary difference between the seismic hazard evaluation methods is that the probabilistic method includes the contribution of hazard from a set of specified seismic sources at their respective distances, while the deterministic approach considers only a selected, generally "worst-case" scenario on the seismic source estimated to pose the greatest hazard to the site. In addition, the probabilistic approach accounts for uncertainties at each step in the analysis and in the final result. In accordance with the requirements of 2001 California Building Code (CBC), a site-specific probabilistic seismic hazard analysis was performed for the site.

PROBABILISTIC HAZARD ANALYSIS:

The probabilistic analysis of seismic hazard is a statistical analysis of seismicity of known regional faults and regional seismic sources attenuated to a particular geographic location. The results of a probabilistic seismic hazard analysis (PSHA) are presented as the annual probability of exceedance of a given strong motion parameter for a particular exposure time (Johnson and others, 1992).

For this report, the seismic hazard analysis computer program EZFRISK, version 7.14 (Risk Engineering, 2006) was used to analyze the location of the site under the criteria for a "very dense soil" site type, equivalent to the 2001 CBC designation, S_C. The estimated value for the peak ground acceleration (PGA) was calculated as the average of the accelerations computed using the attenuation relations of Abrahamson and Silva (1997), Boore et al. (1997), and Sadigh et al. (1997) in relation to seismogenic faults within a 93-mile (150-km) radius of the site. The EZFRISK program considers seismicity from mapped seismogenic faults and background sources (those earthquakes not associated with a mapped fault source) and assumes that the occurrence rate of earthquakes on a fault is proportional to the estimated slip rate of that fault. Potential earthquake magnitudes are correlated to expected seismic sources and the resultant maximum ground acceleration at the site is computed.

Based on the site-specific PSHA performed for the site, the estimated peak horizontal ground acceleration with a 10 percent probability of exceedance in 50 years (statistical return period of 475 years) for a "dense soil" site type S_C, is 0.27g. This value corresponds to the Design Basis Earthquake as defined in the 2001 CBC.

SEISMIC ZONE:

Figure 16-2 presented in the 2001 CBC places the site within Seismic Zone 4. Table 16-I of the 2001 CBC assigns a Seismic Zone Factor "Z" of 0.40 to Seismic Zone 4.

SOIL PROFILE CHARACTERIZATION:

A soil profile type S_C, very dense soil, is appropriate for the site according to the 2001 CBC based on equivalent SPT blowcount data.

NEAR-SOURCE EFFECTS:

The ground shaking hazard to this site is dominated by the Pisgah-Bullion-Mesquite Lake fault system of the ECSZ. The Pisgah fault is located approximately 0.8 kilometer west of the site. The Pisgah-Bullion-Mesquite Lake system is classified as a Type "B" fault by the State of California. The applicable near-source acceleration factor N_A, as defined in the 2001 CBC, is 1.30, and the near-source velocity factor N_V is 1.60.

GROUNDWATER AND LIQUEFACTION

No evidence of springs or perched groundwater conditions was observed on the site during the field investigation or on the aerial photographs reviewed. Groundwater was not encountered in the exploratory borings placed to a maximum depth of 46 feet bgs.

Available groundwater data was reviewed in order to provide an estimate of the historic groundwater conditions for the site. Groundwater data for State Well No. 08N05E01P01S, located approximately 2 miles northwest of the site, indicates a depth to water of 337.5 feet bgs in May 1964 (DWR, 1990). Groundwater data for State Well No. 08N05E10M01S, located approximately 3 1/2 miles west-northwest of the site, indicates a depth to water of 260 feet bgs in May 1964. Dyer and others (1963) report a depth to groundwater of approximately 280 feet bgs in a well located approximately 3 1/3 miles west of the site, and a depth to groundwater of approximately 147 feet in a well located approximately 5 3/4 miles northwest of the site. Dyer and others (1963) also suggest that the Pisgah fault may be a barrier to groundwater in the vicinity of Interstate 40 with deeper groundwater east of the fault. Based on the available data, an historic groundwater high of approximately 260 feet bgs is appropriate for the site and vicinity.

Liquefaction is a process in which strong ground shaking causes saturated soils to lose their strength and behave as a fluid (Matti and Carson, 1991). Ground failure associated with liquefaction can result in severe damage to structures. The geologic conditions for increased susceptibility to liquefaction are: 1) shallow groundwater (generally less than 50 feet in depth); 2) presence of unconsolidated sandy alluvium, typically Holocene in age; and 3) strong ground shaking. All three of these conditions must be present for liquefaction to occur, and only one of these conditions exist on the site. Based upon the depth of historical high groundwater and the relatively high density of the alluvial sediments underlying the site, liquefaction and associated shallow groundwater-related hazards are not anticipated. Further evaluation of the liquefaction potential at the site is not warranted.

SLOPE STABILITY

Natural slopes greater than 5 feet in height or with inclinations exceeding 2(h):1(v) do not exist on the site. It is not anticipated that significant slopes will be necessary for site development. Minor slopes constructed at inclinations not exceeding 2(h):1(v) and less than 10 feet in height should be grossly stable. However, the non-cohesive granular nature of the soils encountered are not conducive to surficial slope stability. It is our recommendation that slope faces be protected from erosion caused by water and wind. Grading plans should be reviewed as to the need for further slope stability analysis.

FLOODING AND EROSION

Evidence of recent flooding of the site was not observed during the field reconnaissance or on the aerial photographs reviewed. However, active drainage channels cross the northern and southern portions of the site. An evaluation of the flood potential of the site falls under the purview of others.

The surficial soils encountered were generally classified as poorly graded sands and silty sands that are moderately susceptible to erosion by wind and water. Positive drainage should be provided, and water should not be allowed to pond on the site. Water should not be allowed to flow over any graded or natural areas in such a way as to cause erosion.

CONCLUSIONS

On the basis of our research, field investigation, and laboratory testing, it is the opinion of this firm that the proposed development is feasible from a geotechnical standpoint, provided the recommendations contained in this report are implemented during planning, grading and construction.

Evidence of faulting on or immediately adjacent to the site was not observed during the geologic field reconnaissance or on the aerial photographs reviewed. The site is not located within an Alquist-Priolo Earthquake Fault Zone.

Moderate seismic shaking of the site can be expected during the lifetime of the proposed structures.

Based on the historic depth of groundwater and dense nature of the sediments beneath the site, liquefaction or other shallow groundwater-related hazards are not anticipated.

Although no evidence of recent flooding was noted in the field or on the aerial photographs reviewed, the potential for such flooding should be evaluated. An evaluation of the flood potential falls under the purview of others.

The relatively planar topography at the site and anticipated development precludes the potential for slope instability at the site. Temporary excavations should conform to State codes with regard to the geologic materials present at the site. Finished slope configurations are not anticipated to be steeper than 2(h):1(v); therefore, slope stability hazards are not anticipated. However, surficial stability is a concern due to the non-cohesive nature of the soils encountered. Protective measures will be necessary to protect the slopes from erosion, especially wind-related.

Based upon our field investigation and test data, it is our opinion that the upper native soils will not, in their present condition, provide uniform or adequate support for the proposed buildings. Our equivalent

SPT data indicated variable in-situ conditions of upper soils, ranging from loose to very dense states. These potentially problematic soils extend approximately 1 to 3 feet below the existing surface elevation. This condition may cause unacceptable differential and/or overall settlement upon application of the anticipated foundation loads. The underlying soils encountered generally consist of poorly graded sand with random strata of silty sand, both with gravel and cobbles, to the maximum depths attained. These soils were generally in dense to very dense states.

The foundations for the proposed buildings should bear on a minimum of 18 inches of properly compacted fill. A compacted fill mat will provide a dense, uniform, high-strength soil layer to distribute the foundation loads over the underlying soils. If localized areas of undocumented fill or loose native soils are encountered, these problematic soils should be removed and replaced with properly compacted fill.

Conventional spread foundations, either individual spread footings and/or continuous wall footings, may be utilized in conjunction with a compacted fill mat.

The dense to very dense native soils underlying the loose upper soils should provide uniform and adequate support for properly designed mirror caissons. It is our understanding that individual mirrors will be supported by single caissons 6 to 8 feet in diameter and approximately 18 feet in depth. Both downward and uplift axial capacities are provided in Appendix "D".

Material was obtained from the existing Pisgah Crater Road for preliminary asphalt concrete pavement design purposes. Preliminary designs are presented in the PRELIMINARY FLEXIBLE PAVEMENT DESIGN section of this Report.

The preliminary chemical/corrosivity tests are provided in the SOIL CORROSION section of this report. Testing for Thermal and Electrical resistivity was performed by our subconsultant, Schiff Associates. A discussion of the test procedures, as well as the results of their tests have been included with their report (Appendix "E").

Provided that the recommendations provided in this report are implemented, construction of the proposed improvements appears to be feasible from geological and geotechnical standpoints, without adversely affecting the adjacent properties.

RECOMMENDATIONS

SEISMIC DESIGN CONSIDERATIONS:

Moderate to severe seismic shaking of the site can be expected during the lifetime of the proposed structures. Therefore, the proposed structures should be designed, constructed, and maintained accordingly.

The ground shaking hazard to this site is dominated by the Pisgah-Bullion-Mesquite Lake fault system of the ECSZ. The applicable near-source acceleration factor N_A, as defined in the 2001 CBC, is 1.30, and the near-source velocity factor N_V is 1.60.

The site is classified as type S_C, very dense soil profile, according to the 2001 CBC.

The following recommendations for grading and design of conventional foundations are followed by recommendations for the installation and design of the proposed caissons.

GENERAL SITE GRADING:

It is imperative that no clearing and/or grading operations be performed without the presence of a representative of the geotechnical engineer. An on-site pre-job meeting with your representatives, the contractor, and the geotechnical engineer/engineering geologist should occur prior to all grading-related operations. Operations undertaken at the site without the geotechnical engineer present may result in exclusions of affected areas from the final compaction report for the project.

Grading of the subject site should be performed, at a minimum, in accordance with these recommendations and with applicable portions of CBC. The following recommendations are presented for your assistance in establishing proper grading criteria.

INITIAL SITE PREPARATION:

All areas to be graded should be stripped of significant vegetation and other deleterious materials. These materials should be removed from the site for disposal. Any existing utility lines should be traced, removed, and rerouted from the structure areas.

Removal of any undocumented fill, as well as the underlying 3 feet of native soils within areas to be graded, including building pad areas and 10 feet beyond, should be conducted in order to help identify any subsurface obstructions or undocumented fills and to remove and recompact the loose upper soils. The engineering geologist should be present during the grading operation to observe and approve open removal excavations prior to scarification and refilling. The purpose of this observation is to verify the presence of competent native material and to identify unsuitable native soil that may extend below the initial removal depth. All such unsuitable material should be removed at that time.

Cavities created by removal of subsurface obstructions such as structures and utility lines should be thoroughly cleaned of loose soil, organic matter, and other deleterious materials, shaped to provide access for construction equipment, and backfilled as recommended for site fill.

PREPARATION OF FILL AREAS:

Prior to placing fill and after observation and approval of the mandatory removal operation, the surfaces of all areas to receive fill should be scarified to a depth of 12 inches or more. The scarified soils should be brought to between optimum moisture content and 2 percent above and recompacted to a minimum relative compaction of 90 percent in accordance with ASTM D 1557.

PREPARATION OF FOOTING AREAS:

The footings of proposed structures should rest upon at least 18 inches of properly compacted fill material. If areas exist where the required thickness of compacted fill is not accomplished by the remedial removals and site rough grading, the footing area excavation should be deepened to provide the recommended fill mat thickness. The subexcavation should extend at least 10 feet beyond the footing lines. Following observation and approval by the engineering geologist, the bottom of this excavation should then be scarified to a depth of at least 12 inches, brought to between optimum moisture content and 2 percent above, and recompacted to a minimum of 90 percent relative compaction in accordance with ASTM D 1557 prior to refilling the excavation to grade as properly compacted fill.

COMPACTED FILLS:

The on-site soils should provide adequate quality fill material provided they are free of organic matter and other deleterious materials. Unless approved by the geotechnical engineer, rock or similar irreducible material with a maximum dimension greater than 3 inches should not be buried or placed in fills.

Import fill should be inorganic, non-expansive granular soil free from rocks or lumps greater than 3 inches in maximum dimension. Sources for import fill should be observed and approved by the geotechnical engineer prior to their use.

Fill should be spread in near-horizontal layers, approximately 8 inches in thickness. Thicker lifts may be approved by the geotechnical engineer if testing indicates that the grading procedures are adequate to achieve the required compaction. Each lift should be spread evenly, thoroughly mixed during spreading to attain uniformity of the material and moisture in each layer, brought to between optimum moisture content and 2 percent above, and compacted to a minimum relative compaction of 90 percent in accordance with ASTM D 1557.

TEMPORARY EXCAVATIONS:

The soils encountered within our exploratory borings are generally classified as a Type "C" soil in accordance with the CAL/OSHA (California, State of, 2001) excavation standards. Unless specifically evaluated by the project engineering geologist, all temporary excavations (in which personnel will enter) should comply with CAL/OSHA (California, State of, 2001) excavation standards for Type "C" soil. Based upon a soil classification of Type "C", the temporary excavations should not be inclined steeper than 1.5(h):1(v) for a maximum depth of 20 feet. For temporary excavations deeper than 20 feet or for conditions that differ from those described for Type "C" in the CAL/OSHA excavation standards, the project geotechnical engineer should be contacted.

FOUNDATION DESIGN:

If the site is prepared as recommended, the proposed buildings may be safely founded on conventional spread foundations, either individual spread footings and/or continuous wall footings, bearing on a minimum of 18 inches of compacted soil. Footings should be a minimum of 12 inches wide and should be established at a minimum depth of 18 inches below lowest adjacent final subgrade level. For the minimum width and depth, footings may be designed for a maximum allowable foundation pressure of 2,100 psf for dead plus live loads. This allowable bearing pressure may be increased by 500 psf for each additional foot of width and by 900 psf for each additional foot of depth to a maximum allowable foundation pressure of 3,500 psf for dead plus live loads. These bearing values may be increased by one-third for wind or seismic loading.

For footings thus designed and constructed, we would anticipate a maximum settlement due to foundation loading of less than 1 inch. Differential settlement between similarly loaded adjacent footings is expected to be approximately one-half the total settlement, not exceeding 1/2 inch in a span of 40 feet.

LATERAL LOADING:

Resistance to lateral loads will be provided by passive earth pressure and base friction. For footings bearing against compacted fill, passive earth pressure may be considered to be developed at a rate of 400 psf per foot of depth. Base friction may be computed at 0.43 times the normal load. Base friction and passive earth pressure may be combined without reduction.

For preliminary retaining wall design purposes, a lateral active earth pressure developed at a rate of 40 psf per foot of depth may be utilized for unrestrained conditions. A lateral at-rest earth pressure developed at a rate of 60 psf per foot of depth should be utilized for restrained conditions. These values should be verified prior to construction when the backfill materials and conditions have been determined and are applicable only to level, properly drained backfill with no additional surcharge loadings.

Foundation concrete should be placed in neat excavations with vertical sides, or the concrete should be formed and the excavations properly backfilled as recommended for site fill.

SLABS-ON-GRADE:

To provide adequate support, concrete slabs-on-grade should bear on a minimum of 18 inches of compacted soil.

The final pad surfaces should be rolled to provide smooth, dense surfaces upon which to place the concrete.

Slabs to receive moisture-sensitive coverings should be provided with a moisture vapor barrier. This barrier may consist of an impermeable membrane. Two inches of sand over the membrane should help to reduce punctures and aid in obtaining a satisfactory concrete cure. The sand should be moistened just prior to placing of concrete.

EXPANSIVE SOILS:

The soils encountered within the exploratory borings were sufficiently granular to preclude a potential for significant expansion. As such, the need for specialized construction procedures to specifically resist expansive soil forces are not anticipated at this time. Requirements for reinforcing steel to satisfy structural criteria are not affected by this recommendation. Additional evaluation of soils for expansion potential should be conducted by the geotechnical engineer during the grading operation.

CAISSON DESIGN RECOMMENDATIONS

According to plans supplied to us, the CIDH piles/caissons will be 6 to 8 feet in diameter. Depth of embedment will be 18 feet. Due to the lack of design information available at the time of this report, we calculated the vertical bearing capacity as a function of shaft diameters, and the lateral capacity as a function of applied shear loads and bending moments. The bending moment acting at the shaft top was assumed to be 15 feet times the shear force acting at the same point. The pile capacities were calculated utilizing a commercial program - Allpile, Verison 7.4i., which in turn directly utilizes COM624S calculation methods for lateral analysis (FHWA-SA-91-048, COM624P – *Laterally Loaded Pile Program for the Microcomputer, Version 2.0*, by Wang and Reese, 1993).

Due to the small aspect ratio of the pile/caisson (L/D), the pile type of "drilled shaft" was selected. AllPile (ver 7.4i) uses the procedures described in "Drilled Shafts: Construction Procedures and Design Methods (FHWA-IF-99-025, 1997)." Therefore, the term "shaft" is used hereafter.

Please note that design loads were not available at the time of preparation of this report. The recommended design parameters below should be reviewed and revised as necessary when design loads and loading configurations are available.

ALLOWABLE AXIAL SHAFT CAPACITIES:

Both upward and downward allowable axial capacities were calculated for 72, 84, and 96-inch diameter concrete CIDH shafts. The embedment depths were taken as 18 feet and should be measured from the bottom of the shaft cap, which has been assumed to be approximately 3 feet below the finish grade of the pad or ground surface. Greater or lesser pile cap elevations should result in a corresponding decrease or increase in shaft depth.

The recommended capacities apply to the total of dead plus live loads and are gross values at the pile head. Both ultimate and allowable capacities are presented in Table 1. The design engineer should select capacities according to the design method selected. If the "strength design" method is selected, ultimate capacities should be utilized. Alternatively, if the "allowable stress design" method is used, allowable capacities should be selected.

Ultimate and allowable vertical capacity vs. pile length for both downward and uplift capacities are included in Enclosures "D-1" through "D-3" for shaft diameters of 72, 84, and 96 inches respectively.

The maximum allowable downward capacity includes a factor of safety of 2.0 for skin friction and 2.5 for tip bearing. The maximum allowable uplift capacity includes a factor of safety of 3.0 for skin friction and 1.0 for pile weight. Utilizing these values, the combined dead plus live loads should be limited to the values presented in Table 1. We have also included the ultimate downward capacities for the shafts should other factors of safety be desired. These capacities may be increased by one-third for wind or seismic loading. The capacities provided are based on soil strengths. Structural capacities of piles must be verified by the design engineer.

The pile length shown in Table 1 is based on the assumption that the top of the shaft will be approximately 3 feet bgs. It should also be noted that practical refusal may be achieved prior to reaching the minimum depth of embedment. Stopping the pile short of the minimum depth of embedment will reduce pile capacity during a seismic event.

For a properly installed shaft, it is anticipated that a total settlement between 1 and 1 1/2 inches per shaft will be required to fully mobilize the indicated capacities, as shown in the following table. Settlemtn of shafts for lesser loads can be linearly interpreted, i.e. for a 72-inch diameter shaft with a load of 440 kips, the estimated settlement is 1/2 inch.

TABLE 1
AXIAL AND LATERAL SHAFT CAPACITIES

Pile Diameter (in.)	Length of Pile (ft.)	Ultimate Downward Capacity (kips)	Ultimate Uplift Capacity (kips)	Allowable Downward Capacity (kips)	Allowable Uplift Capacity (kips)	Estimated Settlement (in.)
72	18	2150	440	880	200	1.0
84	18	2840	530	1160	250	1.5
96	18	3620	620	1470	300	1.5

LATERAL SHAFT ANALYSES:

As part of our lateral shaft capacity evaluations, we analyzed the behavior of 72, 84, and 96-inch drilled shafts. A combined shear force and bending moment was assumed to act on the top of the shaft. The bending moment was assumed as 15 feet times the shear force. The graphed results, showing lateral load vs. head deflection and lateral load vs. maximum moment, and the distribution of deflection, moment, and shear force with embedded depth are included in Enclosures "E-4" through "E-9". Enclosure "D-10" summarizes the relationships between shaft top deflections and lateral loads. The design engineer should utilize this chart to obtain lateral capacity for his design deflection (allowable deflection).

An elastic modulus (E) of 3,000 kip/in^2 was utilized for shaft material (concrete) in our calculation.

DRILLED SHAFT INSTALLATION:

The installation of the drilled shafts should be observed by the geotechnical engineer to verify the soil condition and that the desired diameter and depth of pile are achieved. The drilled shafts should be true and plumb. Because of the granular non-cohesive nature and gravel/cobble content of the soils encountered it is anticipated that caving will likely occur during drilling or during construction of the shafts. As such, appropriate measures will be required to minimize caving. It is recommended that a test boring of the same diameter as the proposed shafts be placed prior to the beginning of construction. The contractor performing the work should have experience in this type of soil and construction.

CIDH excavations should be filled with concrete as soon after drilling as possible. In no event should pile holes be left open over night. The concrete should be placed so that the concrete is not allowed to fall freely more than 5 feet and is prevented from striking the walls of the shaft or the reinforcement bars, thus causing caving and/or segregation of the concrete. All loose materials should be cleared from the bottom of the pile excavation. If casing is necessary and is utilized, then the casing should be withdrawn concurrently with the concrete placement.

PRELIMINARY FLEXIBLE PAVEMENT DESIGN:

Based upon our preliminary sampling and testing (R-value of 60), and upon an assumed traffic index (T.I.) of 8.0 for roadways and 6.0 for automobile parking areas, it appears that the structural sections tabulated below should provide satisfactory asphalt concrete (AC) pavement.

Area	T.I.	Recommended Street Section
Pisgah Crater Road	8.0	0.40' AC/0.35' AB Class 2 or 0.55' AC/Compacted Native
Parking Areas	6.0	0.25' AC/0.35' AB Class 2 or 0.35' AC/Compacted Native

AB = Aggregate Base

The above structural sections are predicated upon proper compaction of the utility trench backfills and the subgrade soils, with the upper 6 inches of subgrade soils and all AB material brought to a relative compaction of at least 95 percent in accordance with ASTM D 1557 prior to paving. The AB should meet Caltrans requirements for Class 2 base.

It should be noted that the above preliminary pavement designs were based upon the results of preliminary sampling and testing performed on this project and should be verified by additional sampling and testing during construction when the actual subgrade soils are exposed.

C.H.J., Incorporated does not practice traffic engineering. The T.I.s used to develop the recommended pavement sections are typical for projects of this type. We recommend that the T.I.s used be reviewed by the project civil engineer or traffic engineer to verify that they are appropriate for this project.

SOIL CORROSION:

Selected samples of material were delivered to our subconsultant, Schiff Associates, for soil corrosivity tests. Laboratory testing consisted of pH, resistivity, and major soluble salts commonly found in soils. The results of the laboratory tests performed by our consultant are enclosed. These tests have been performed in order to screen the site for potentially corrosive soils.

Values obtained from the testing indicate that soils are considered mildly corrosive at as-received conditions, and corrosive to severely corrosive at saturated moisture conditions to ferrous metals at the site.

Results of the soluble sulfate testing indicate a "negligible" anticipated exposure to sulfate attack, as indicated on the enclosed test results. Based upon the criteria from Table 4.3.1. of the American Concrete Institute Manual of Concrete Practice (2000), no special measures, such as specific cement types, water-cement ratios, etc., will be needed for this "negligible" exposure to sulfate attack.

Soluble chloride content of soil was not at levels high enough to be of concern with respect to corrosion of reinforcing steel. The results should be considered in combination with the soluble chloride content of the hardened concrete in determining the effect of chloride on the corrosion of reinforcing steel.

Ammonium content of soil was not at a sufficiently high level to be of concern with respect to corrosion of buried copper. Nitrate content was at a borderline level where corrosion of buried copper begins to be of concern.

C.H.J., Incorporated does not practice corrosion engineering. If further information concerning the corrosion characteristics, or interpretation of the results submitted herein, are required, then a competent corrosion engineer could be consulted.

CONSTRUCTION OBSERVATION:

All earthwork operations, including site clearing, stripping, grading, and caisson excavation, should be observed by a representative of the geotechnical engineer. The presence of the geotechnical engineer's field representative will be for the purpose of providing observation and field testing, and will not include any supervising or directing of the actual work of the contractor, his employees, or agents. Neither the presence of the geotechnical engineer's field representative nor the observations and testing by the geotechnical engineer shall excuse the contractor in any way for defects discovered in his work. It is understood that the geotechnical engineer will not be responsible for job or site safety on this project, which will be the sole responsibility of the contractor.

LIMITATIONS

C.H.J., Incorporated has striven to perform our services within the limits prescribed by our client, and in a manner consistent with the usual thoroughness and competence of reputable geotechnical engineers and engineering geologists practicing under similar circumstances. No other representation, express or implied, and no warranty or guarantee is included or intended by virtue of the services performed or reports, opinion, documents, or otherwise supplied.

This report reflects the testing conducted on the site as the site existed during the investigation, which is the subject of this report. However, changes in the conditions of a property can occur with the passage of time, due to natural processes or the works of man on this or adjacent properties. Changes in applicable or appropriate standards may also occur whether as a result of legislation, application, or the broadening of knowledge. Therefore, this report is indicative of only those conditions tested at the time of the subject investigation, and the findings of this report may be invalidated fully or partially by changes outside of the control of C.H.J., Incorporated. This report is therefore subject to review and should not be relied upon after a period of one year.

The conclusions and recommendations in this report are based upon observations performed and data collected at separate locations, and interpolation between these locations, carried out for the project and the scope of services described. It is assumed and expected that the conditions between locations observed and/or sampled are similar to those encountered at the individual locations where observation and sampling was performed. However, conditions between these locations may vary significantly. Should conditions be encountered in the field, by the client or any firm performing services for the client or the client's assign, that appear different than those described herein, this firm should be contacted immediately in order that we might evaluate their effect.

If this report or portions thereof are provided to contractors or included in specifications, it should be understood by all parties that they are provided for information only and should be used as such.

The report and its contents resulting from this investigation are not intended or represented to be suitable for reuse on extensions or modifications of the project, or for use on any other project.

CLOSURE

We appreciate this opportunity to be of service and trust this report provides the information desired at this time. Should questions arise, please do not hesitate to contact this office.

PROFESSIONAL GEOLOGIST
BENNIE DON WILLIAMS
NO. 7542
EXP. 1-31-07
STATE OF CALIFORNIA
11-6-06

Respectfully submitted,
C.H.J., INCORPORATED

Mack Chen
Staff Geologist

Ben Williams. P.G. 7542
Senior Staff Geologist

REGISTERED GEOLOGIST
JAY J. MARTIN
No. 1529
CERTIFIED ENGINEERING GEOLOGIST
Exp. 8-31-08
STATE OF CALIFORNIA
11-6-06

Jay J. Martin, E.G. 1529
Vice President

Allen D. Evans, G.E. 2060
Vice President

REGISTERED PROFESSIONAL ENGINEER
ALLEN D. EVANS
No. 2060
Exp. 3-31-2007
GEOTECHNICAL
STATE OF CALIFORNIA
11-6-06

MC/BW/JJM/ADE:sra

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APPENDIX "A"

GEOTECHNICAL MAPS

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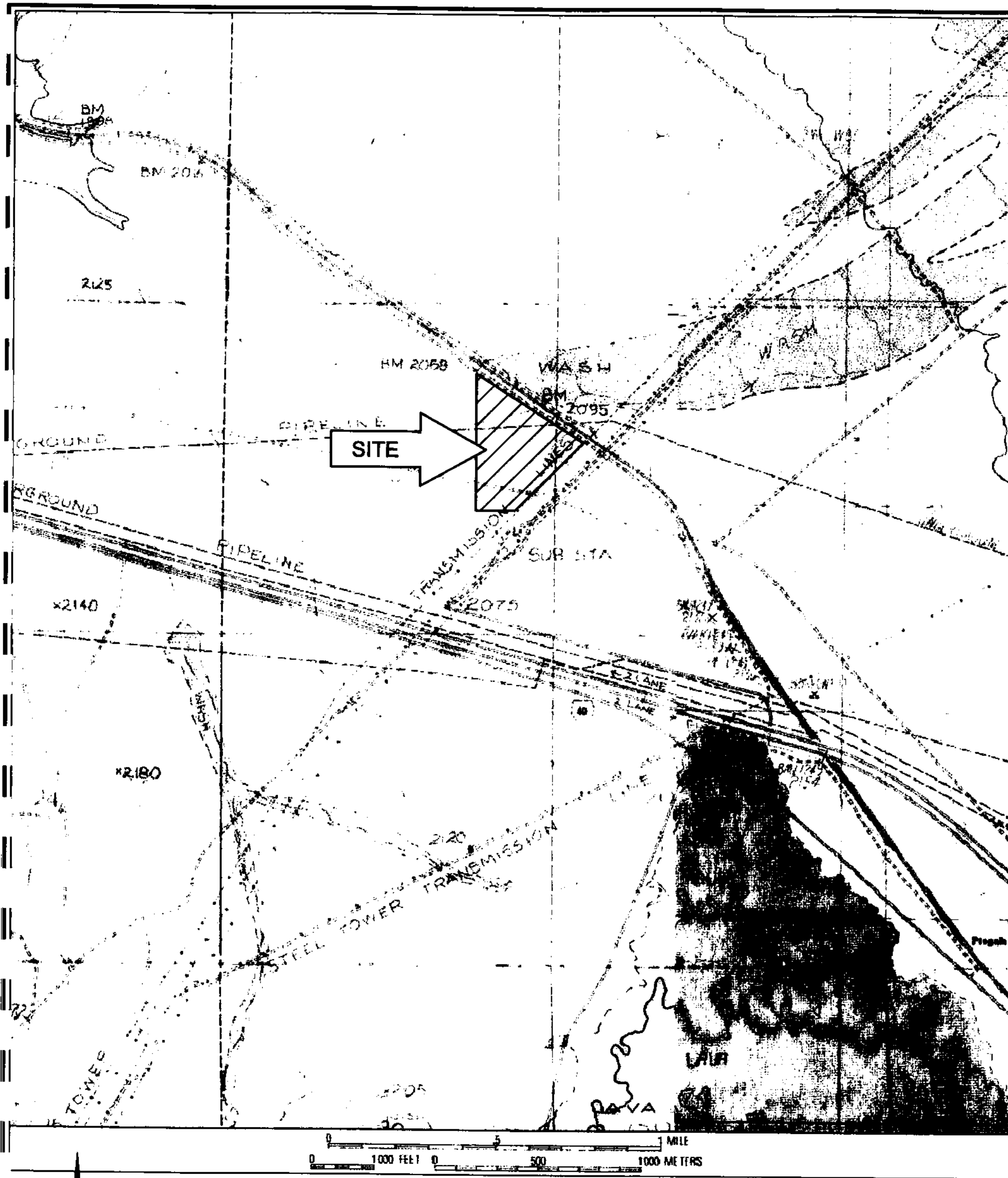


SCALE: 1"= 2,000'

INDEX MAP

FOR: STIRLING ENERGY SYSTEMS, INCORPORATED

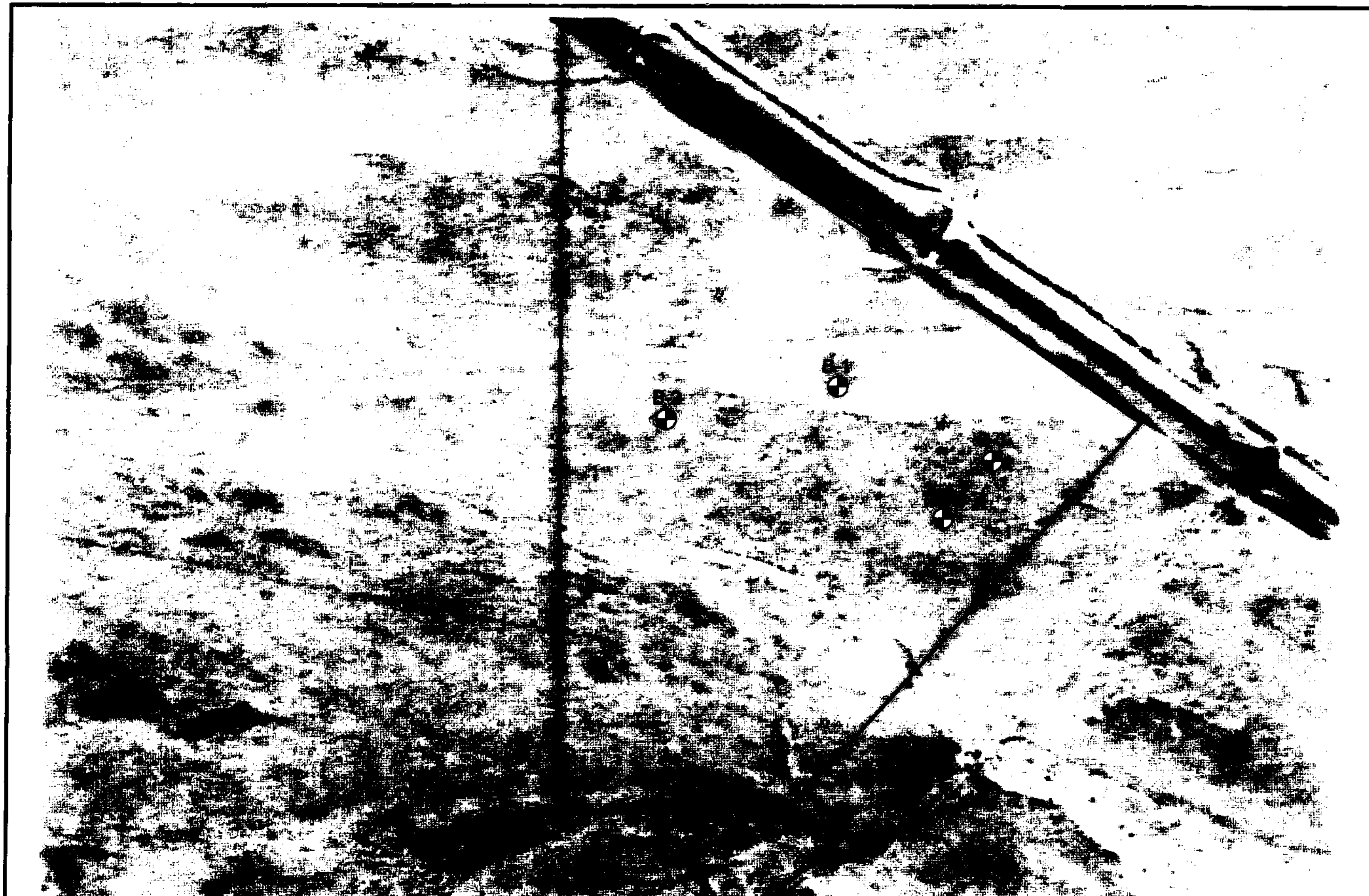
DATE: OCTOBER 2006

GEOTECHNICAL INVESTIGATION
PROPOSED SOLAR ONE-DEMONSTRATION SITE
NEWBERRY SPRINGS/LUDLOW AREA
SAN BERNARDINO COUNTY, CALIFORNIA

ENCLOSURE "A-1"

JOB NUMBER V06100-3

C.H.J. Incorporated



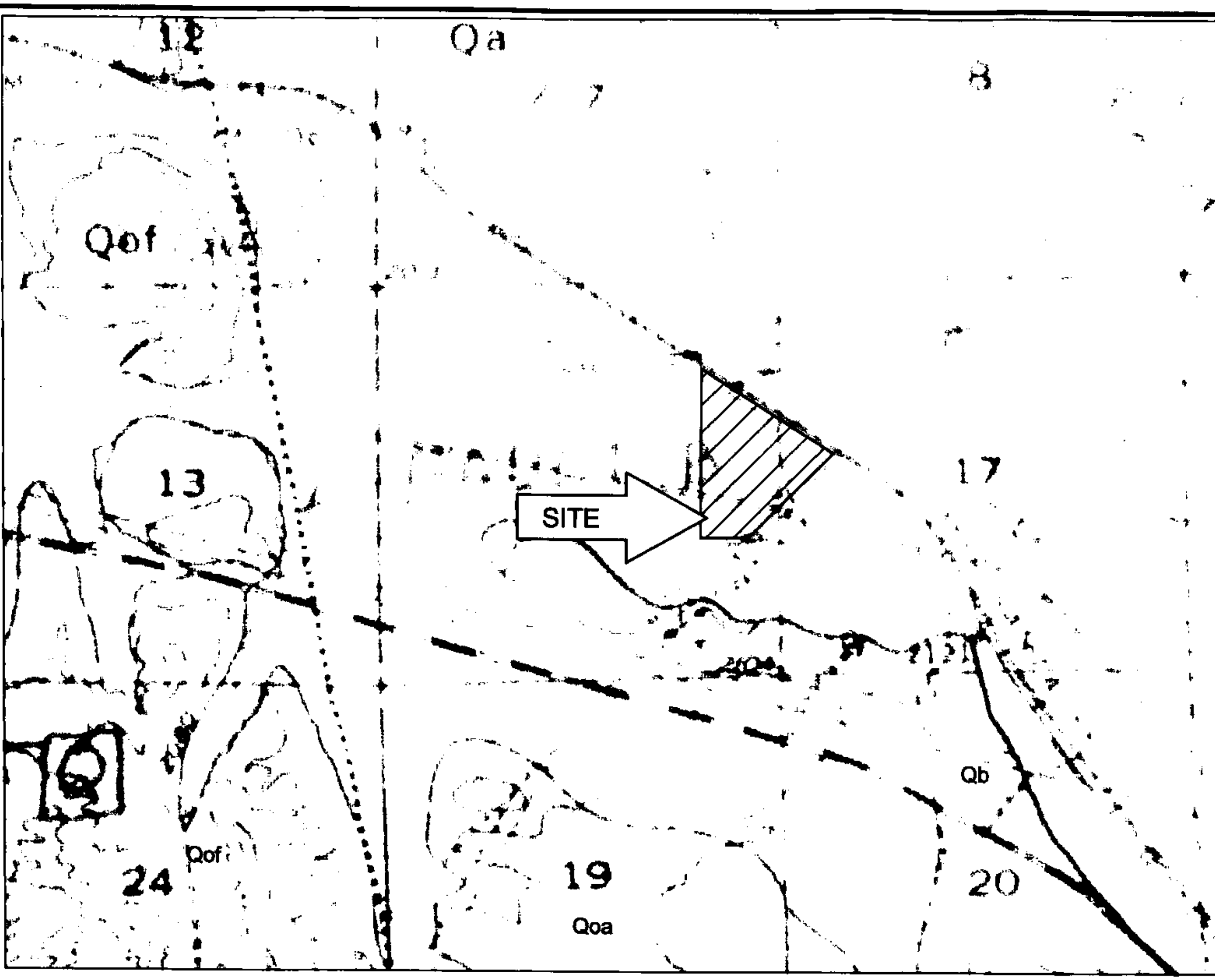
LEGEND:

B-4 BORING LOCATIONS

PLAT

FOR: STIRLING ENERGY SYSTEMS, INCORPORATED	GEOTECHNICAL INVESTIGATION PROPOSED SOLAR ONE-DEMONSTRATION SITE NEWBERRY SPRINGS/LUDLOW AREA SAN BERNARDINO COUNTY, CALIFORNIA
DATE: OCTOBER 2006	ENCLOSURE "A-2" JOB NUMBER V06100-3

C.H.J. Incorporated



GEOLOGIC UNITS

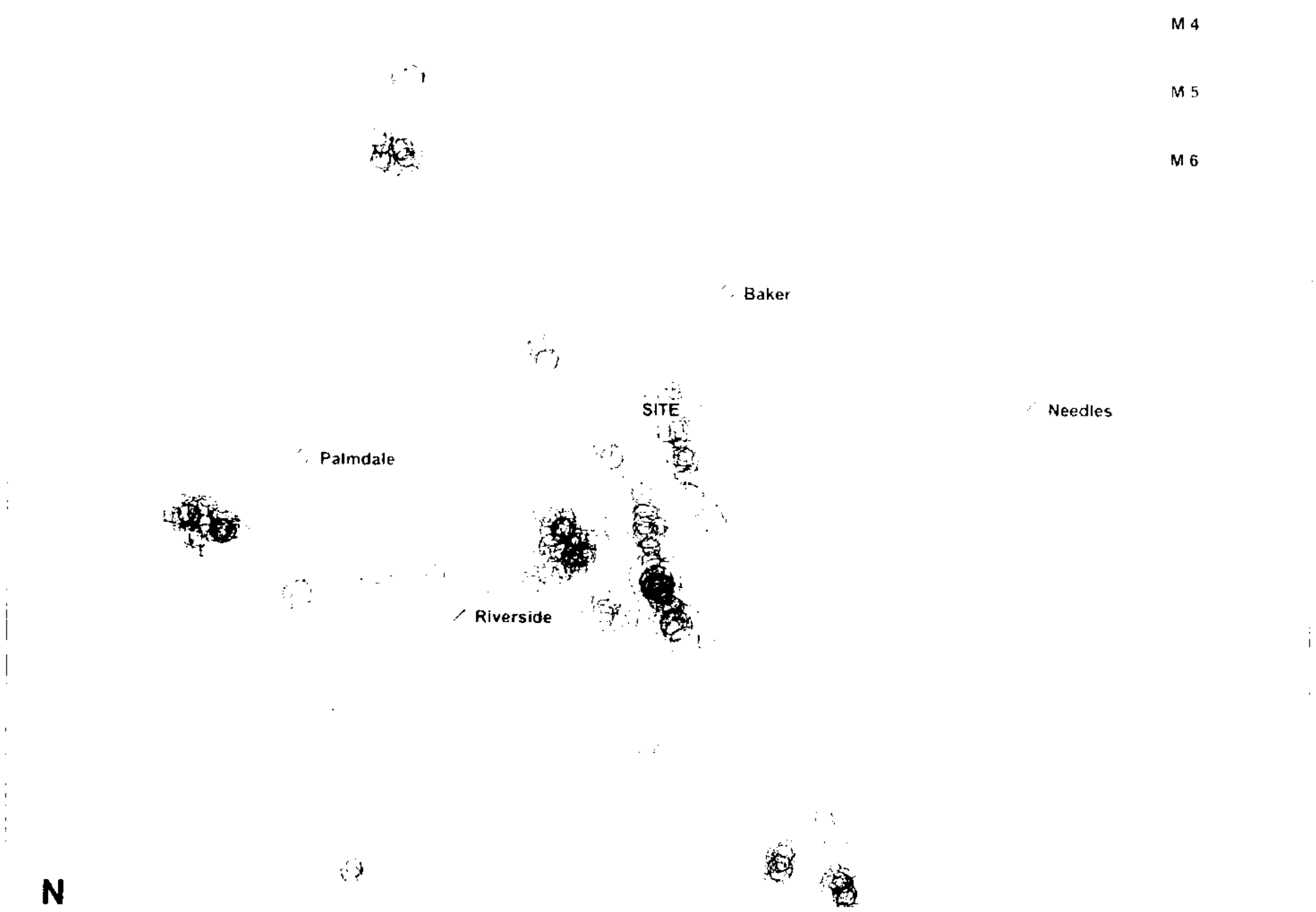
N

SCALE: 1"= 2,000'

GEOLOGIC INDEX MAP

FOR: STIRLING ENERGY SYSTEMS, INCORPORATED	GEOTECHNICAL INVESTIGATION PROPOSED SOLAR ONE-DEMONSTRATION SITE NEWBERRY SPRINGS/LUDLOW AREA SAN BERNARDINO COUNTY, CALIFORNIA	ENCLOSURE "A-3"
DATE: OCTOBER 2006		JOB NUMBER V06100-3

C.H.J. Incorporated



Seismicity 1977-2005 (Magnitude 4.0+) 150 kilometer radius

SITE LOCATION: 34.78588 LAT. -116.38775 LONG.

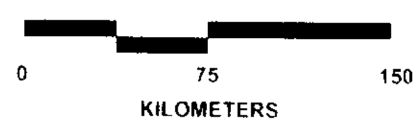
MINIMUM LOCATION QUALITY: C

TOTAL # OF EVENTS ON PLOT: 658

TOTAL # OF EVENTS WITHIN SEARCH RADIUS 349

MAGNITUDE DISTRIBUTION OF SEARCH RADIUS EVENTS:

4.0- 4.9 : 314
5.0- 5.9 : 31
6.0- 6.9 : 2
7.0- 7.9 : 2
8.0- 8.9 : 0



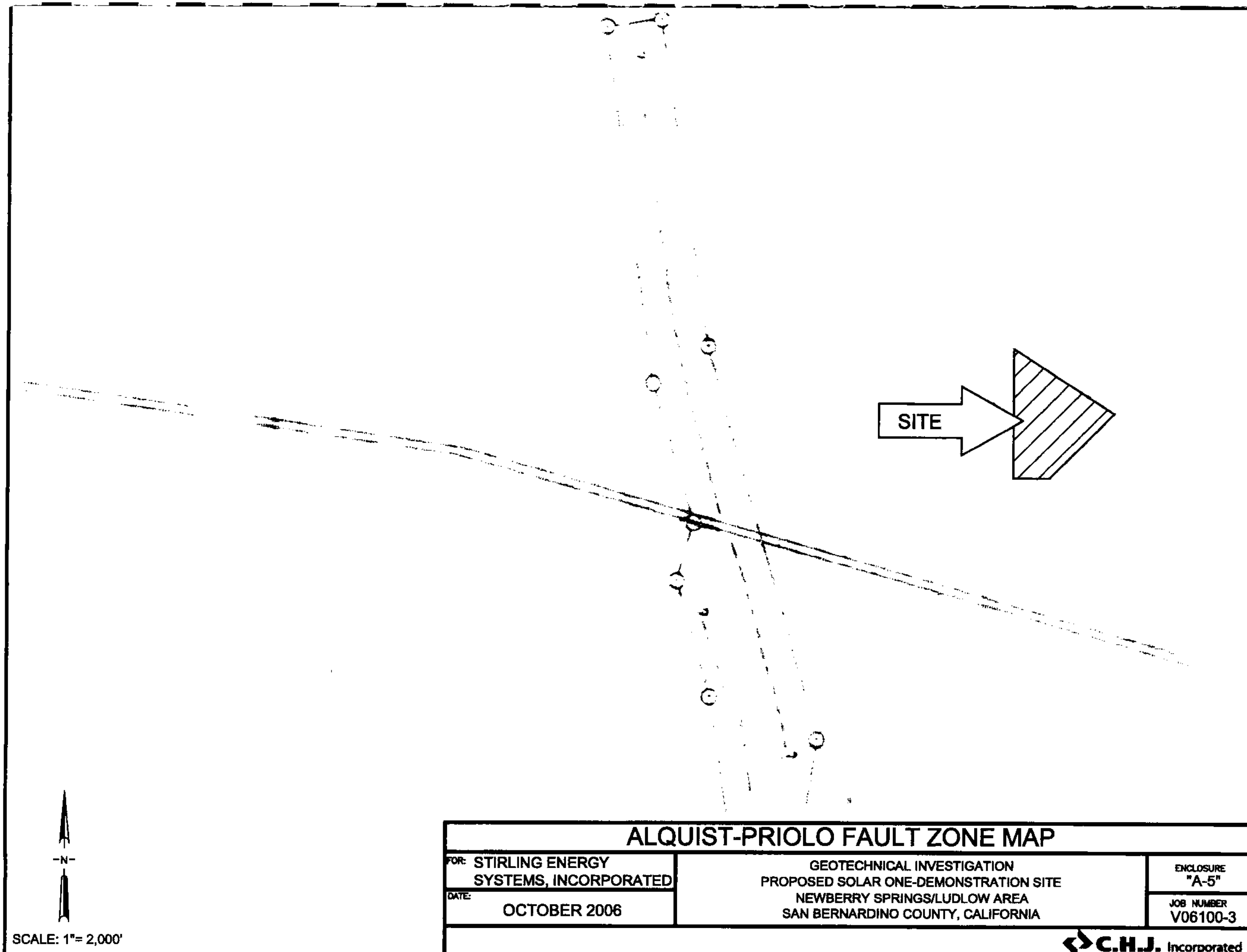
CLOSEST EVENT: 4.4 ON MONDAY, FEBRUARY 14, 2000 LOCATED APPROX 2 KILOMETERS NORTHEAST OF THE SITE

LARGEST 5 EVENTS:

7.3 ON SUNDAY, JUNE 28, 1992 LOCATED APPROX 65 KILOMETERS SOUTH OF THE SITE
7.1 ON SATURDAY, OCTOBER 16, 1999 LOCATED APPROX 23 KILOMETERS SOUTHEAST OF THE SITE
6.4 ON SUNDAY, JUNE 28, 1992 LOCATED APPROX 76 KILOMETERS SOUTHWEST OF THE SITE
6.1 ON THURSDAY, APRIL 23, 1992 LOCATED APPROX 91 KILOMETERS SOUTH OF THE SITE
5.8 ON SATURDAY, OCTOBER 16, 1999 LOCATED APPROX 14 KILOMETERS SOUTHEAST OF THE SITE

EARTHQUAKE EPICENTER MAP		
FOR: STIRLING ENERGY SYSTEMS, INCORPORATED	GEOTECHNICAL INVESTIGATION PROPOSED SOLAR ONE-DEMONSTRATION SITE NEWBERRY SPRINGS/LUDLOW AREA SAN BERNARDINO COUNTY, CALIFORNIA	ENCLOSURE "A-4"
DATE: OCTOBER 2006		JOB NUMBER V06100-3

C.H.J. Incorporated



ALQUIST-PRIOLO FAULT ZONE MAP

FOR: STIRLING ENERGY SYSTEMS, INCORPORATED	GEOTECHNICAL INVESTIGATION PROPOSED SOLAR ONE-DEMONSTRATION SITE NEWBERRY SPRINGS/LUDLOW AREA SAN BERNARDINO COUNTY, CALIFORNIA	ENCLOSURE "A-5"
DATE: OCTOBER 2006		JOB NUMBER V06100-3

C.H.J. Incorporated

APPENDIX "B"

EXPLORATORY LOGS

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KEY TO LOGS

LEGEND OF LAB/FIELD TESTS:

Bulk	Indicates Disturbed or Bulk Sample
Cor.	Chemical/Corrosivity Tests
Dist.	Indicates Disturbed Sample
DS	Direct Shear Test (ASTM D 3080)
MDC	Maximum Density Optimum Moisture Determination (ASTM D 1557)
N.R.	Indicates No Recovery of Sample
Ring	Indicates Undisturbed Ring Sample. Undisturbed Ring Samples are obtained with a "California Sampler" (3.25" O.D. and 2.42" I.D.) driven with a 140-pound weight falling 30 inches. The blows per foot are converted to equivalent SPT values.
SA	Sieve Analysis (ASTM C 136)
SPT	Indicates Standard Penetration Test. The SPT N-value is the number of blows required to drive an SPT sampler 12 inches using a 140-pound weight falling 30 inches. The SPT sampler is 2" O.D. and 1 3/8" I.D.

ENGINEERING PROPERTIES FROM SPT BLOWS

Relationship of Penetration Resistance to Relative Density for Cohesionless Soils*
(After Mitchell and Katti, 1981)

Number of SPT Blows (N_{60})	Descriptive Relative Density	Approximate Relative Density (%)
<4	Very Loose	0-15
4-10	Loose	15-35
10-30	Medium Dense	35-65
30-50	Dense	65-85
>50	Very Dense	85-100

* At an effective overburden pressure of 1 ton per square foot (100 kPa)

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			GRAPH SYMBOL	LETTER SYMBOL	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
	MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
	SAND AND SANDY SOILS	CLEAN SAND (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
				SP	POORLY-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	MORE THAN 50% OF COARSE FRACTION PASSING NO. 4 SEIVE	SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SITLY SANDS, SAND-SILT MIXTURES
				SC	CLAYEY SANDS, SAND-CLAY MIXTURES
FINE GRANED SOILS	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTY, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASICITY, FAT CLAYS
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

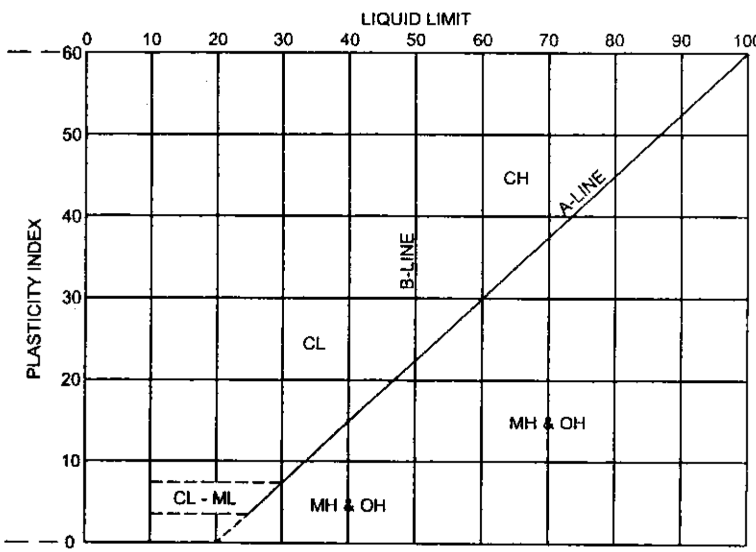
FOR LABORATORY CLASSIFICATION OF FIN-GRAINED SOILS

GRADATION CHART

MATERIAL SIZE	PARTICLE SIZE			
	LOWER LIMIT		UPPER LIMIT	
	MILLIMETERS	SIEVE SIZE	MILLIMETERS	SIEVE SIZE
SAND				
FINE	.074	#200x	0.42	#40 x
MEDIUM	042	#40 x	2.00	#10 x
COARSE	200	#10 x	4.76	# 4 x
GRAVEL				
FINE	4.76	# 4 X	191	3/4"•
COARSE	191	3/4"•	76 2	3" •
COBBLES	76.2	3"	304.8	12"
BOULDERS	304.8	12	914.4	36"

x US STANDARD • CLEAR SQUARE OPENINGS

PLASTICITY CHART



UNIFIED SOIL CLASSIFICATION SYSTEM

C.H.J. Incorporated

EXPLORATORY BORING NO. 1

Date Drilled: 9/26/06 | Client: Stirling Energy Systems, Inc.

Equipment: CME 75 Drill Rig | Driving Weight / Drop: 140 lbs / 30 in

Surface Elevation (Ft.): 0556327E / 3849532N | Logged by: S.C. | Measured Depth to Water(ft): N/A

DEPTH (ft)	GRAPHIC LOG	VISUAL CLASSIFICATION	REMARKS	SAMPLES DRIVE	SAMPLES BULK	BLOWS/FOOT (Equiv. SPT)	FIELD MOISTURE (%)	DRY UNIT WT. (pcf)	LAB/FIELD TESTS
0		(SM) Silty Sand, fine to medium with coarse and gravel to 2 1/2", brown	Native		X		2.9		
2				X		39			SPT
5				X		30/5"			SPT
10				X		72			SPT
15				X		116			SPT
20				X		106			SPT
25				X		112			SPT
29		END OF BORING NO BEDROCK REFUSAL AT 29.0' NO FILL HEAVY CAVING NO FREE GROUNDWATER	Refusal						
35									
40									
45									

BORING_LOG_50_FT V06100-3.GPJ CHJ.GDT 10/31/06

EXPLORATORY BORING NO. 2

Date Drilled: 9/26/06 | Client: Stirling Energy Systems, Inc.

Equipment: CME 75 Drill Rig | Driving Weight / Drop: 140 lbs / 30 in

Surface Elevation (Ft.): 0556164E / 3849494N | Logged by: S.C. | Measured Depth to Water(ft): N/A

DEPTH (ft)	GRAPHIC LOG	VISUAL CLASSIFICATION	REMARKS	SAMPLES DRIVE	SAMPLES BULK	BLOWS/FOOT (Equiv. SPT)	FIELD MOISTURE (%)	DRY UNIT WT. (pcf)	LAB/FIELD TESTS
		(SM) Silty Sand, fine to medium with coarse and gravel to 2 1/2", brown	Native		X		2.8		DS, MDC, SA
				X		30/6"	4.7	Dist.	Ring
5									
				X		30/1"	2.5	Dist.	Ring
10									
				X		30/6"	N.R.	N.R.	Ring
15									
				X		30/6"	N.R.	N.R.	Ring
20									
				X		30/5"	N.R.	N.R.	Ring
25									
				X		30/5"	N.R.	N.R.	Ring
30									
				X		30/6"	N.R.	N.R.	Ring
35		END OF BORING NO BEDROCK NO REFUSAL NO FILL HEAVY CAVING NO FREE GROUNDWATER							
40									
45									

BORING_LOG_50_FT V06100-3.GPJ CHJ.GDT 10/31/06

EXPLORATORY BORING NO. 3

Date Drilled: 9/26/06 | Client: Stirling Energy Systems, Inc.

Equipment: CME 75 Drill Rig | Driving Weight / Drop: 140 lbs / 30 in

Surface Elevation (Ft.): 0556479E / 3849457N | Logged by: S.C. | Measured Depth to Water(ft): N/A

DEPTH (ft)	GRAPHIC LOG	VISUAL CLASSIFICATION	REMARKS	SAMPLES DRIVE	SAMPLES BULK	BLOWS/FOOT (Equiv. SPT)	FIELD MOISTURE (%)	DRY UNIT WT. (pcf)	LAB/FIELD TESTS
0		(SM) Silty Sand, fine to medium, light brown	Native		X		1.3		DS, MDC, SA
				X		10			SPT
3		(SM) Silty Sand, fine to medium with coarse and gravel to 1 1/2", brown							
5				X		42			SPT
10				X		42			SPT
15				X		46			SPT
20				X		56			SPT
25				X		59			SPT
30				X		109/10"			SPT
35				X		115/10"			SPT
40				X		116/11"			SPT
45				X		70/3"			SPT
		END OF BORING NO BEDROCK, NO REFUSAL, NO FILL HEAVY CAVING, NO FREE GROUNDWATER							

BORING_LOG_50_FT V06100-3.GPJ CHJ.GDT 10/31/06

C.H.J.

SOLAR ONE-DEMONSTRATION SITE
SAN BERNARDINO COUNTY, CALIFORNIA

Job No. V06100-3

Enclosure B-3

EXPLORATORY BORING NO. 4

Date Drilled: 9/26/06 | Client: Stirling Energy Systems, Inc.

Equipment: CME 75 Drill Rig | Driving Weight / Drop: 140 lbs / 30 in

Surface Elevation (Ft.): 0556431E / 3849407N | Logged by: S.C. | Measured Depth to Water(ft): N/A

DEPTH (ft)	GRAPHIC LOG	VISUAL CLASSIFICATION	REMARKS	SAMPLES DRIVE	SAMPLES BULK	BLOWS/FOOT (Equiv. SPT)	FIELD MOISTURE (%)	DRY UNIT WT. (pcf)	LAB/FIELD TESTS
		(SM) Silty Sand, fine to medium with coarse and gravel to 1 1/2", brown	Native		X		2.7		
				X		75			SPT
5									
				X		60			SPT, SA
10									
				X		112			SPT, SA
15									
				X		50			SPT
20									
				X		48			SPT
25									
				X		73			SPT
30									
				X		70/3"			SPT
35		END OF BORING							
		NO BEDROCK NO REFUSAL NO FILL HEAVY CAVING NO FREE GROUNDWATER							
40									
45									

BORING_LOG_50_FT V06100-3.GPJ CHJ.GDT 10/31/06

APPENDIX "C"

LABORATORY TESTING

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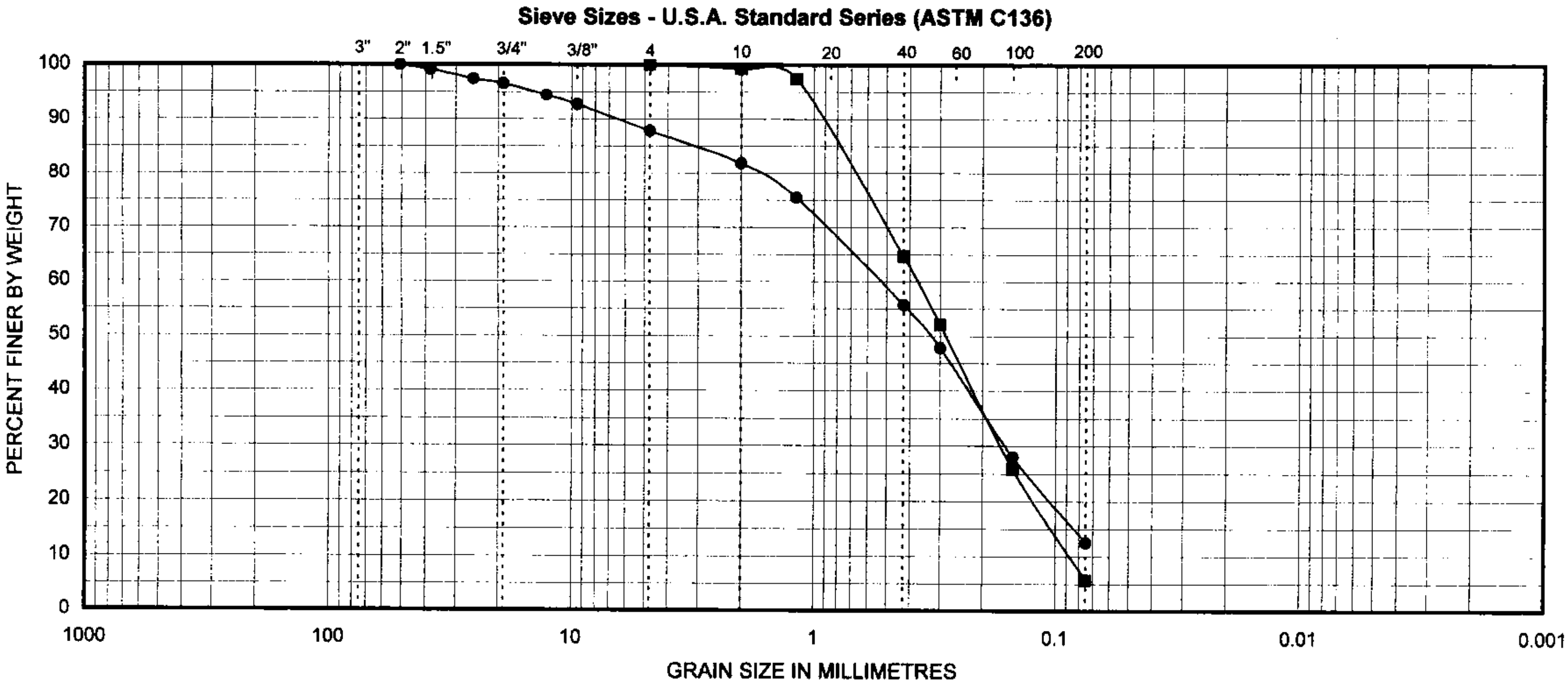


Cobbles & Boulders	Gravel		Sand			Silt	Clay
	Coarse	Fine	Coarse	Medium	Fine		

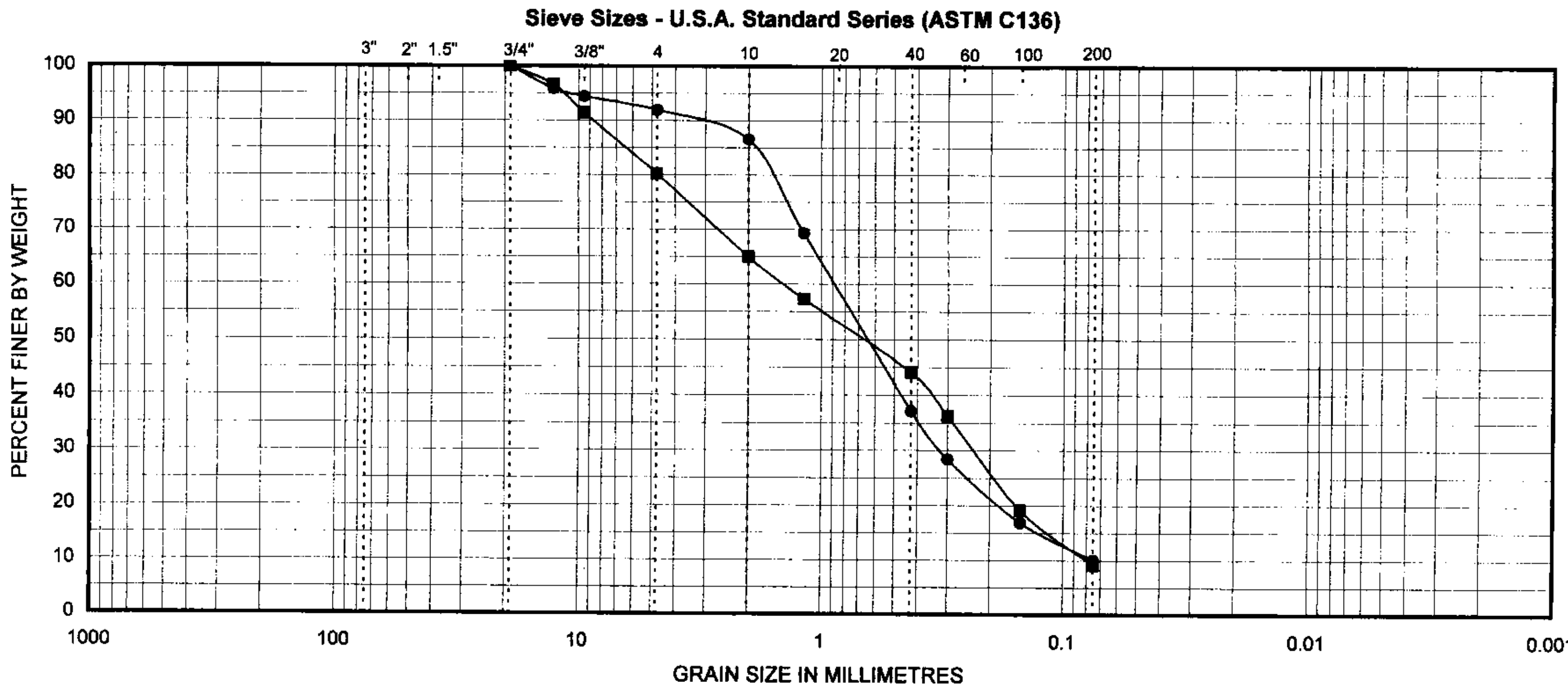
Symbol	Boring No.	Depth (ft)	Classification	D_{10} (mm)	D_{30} (mm)	D_{50} (mm)	D_{60} (mm)	C_u	C_c	SE
●	2	0	(SM) Silty Sand, fine to coarse with gravel to 2"		0.161	0.330	0.531			
■	3	0	(SP-SM) Sand, fine to medium with silt	0.0871	0.168	0.284	0.374	4.293	0.863	

C.H.J. Incorporated

GRADATION CURVES

Project:	Solar One Demonstration Site		
Location:	Newberry Springs/Ludlow Area, San Bernardino County, CA		
Job Number:	V06100-3	Enclosure:	C-1

CHJ® LabSuite ver2.15. Programmed by Dr. Fred Yi



Cobbles & Boulders	Gravel		Sand			Silt	Clay
	Coarse	Fine	Coarse	Medium	Fine		

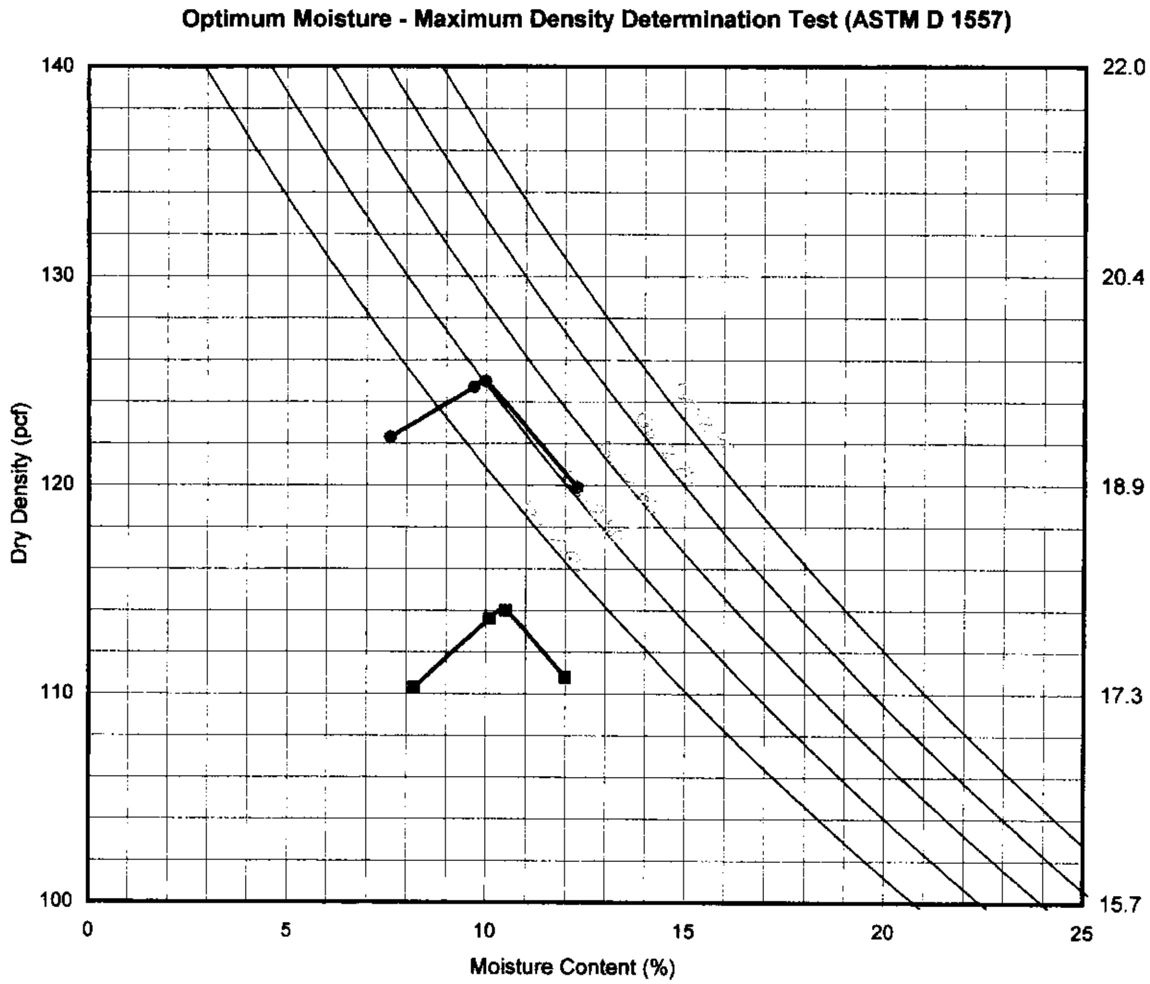
Symbol	Boring No.	Depth (ft)	Classification	D_{10} (mm)	D_{30} (mm)	D_{50} (mm)	D_{60} (mm)	C_u	C_c	SE
●	4	7.5	(SP-SM) Sand, fine to medium with coarse and gravel to 3/4"		0.320	0.640	0.880			
■	4	12.5	(SP-SM) Sand, fine to coarse; Gravel to 3/4"	0.0798	0.233	0.674	1.424	17.828	0.477	

C.H.J. Incorporated

GRADATION CURVES

Project:	Solar One Demonstration Site		
Location:	Newberry Springs/Ludlow Area, San Bernardino County, CA		
Job Number:	V06100-3	Enclosure:	C-2

CHJ® LabSuite ver2.15. Programmed by Dr. Fred Yi

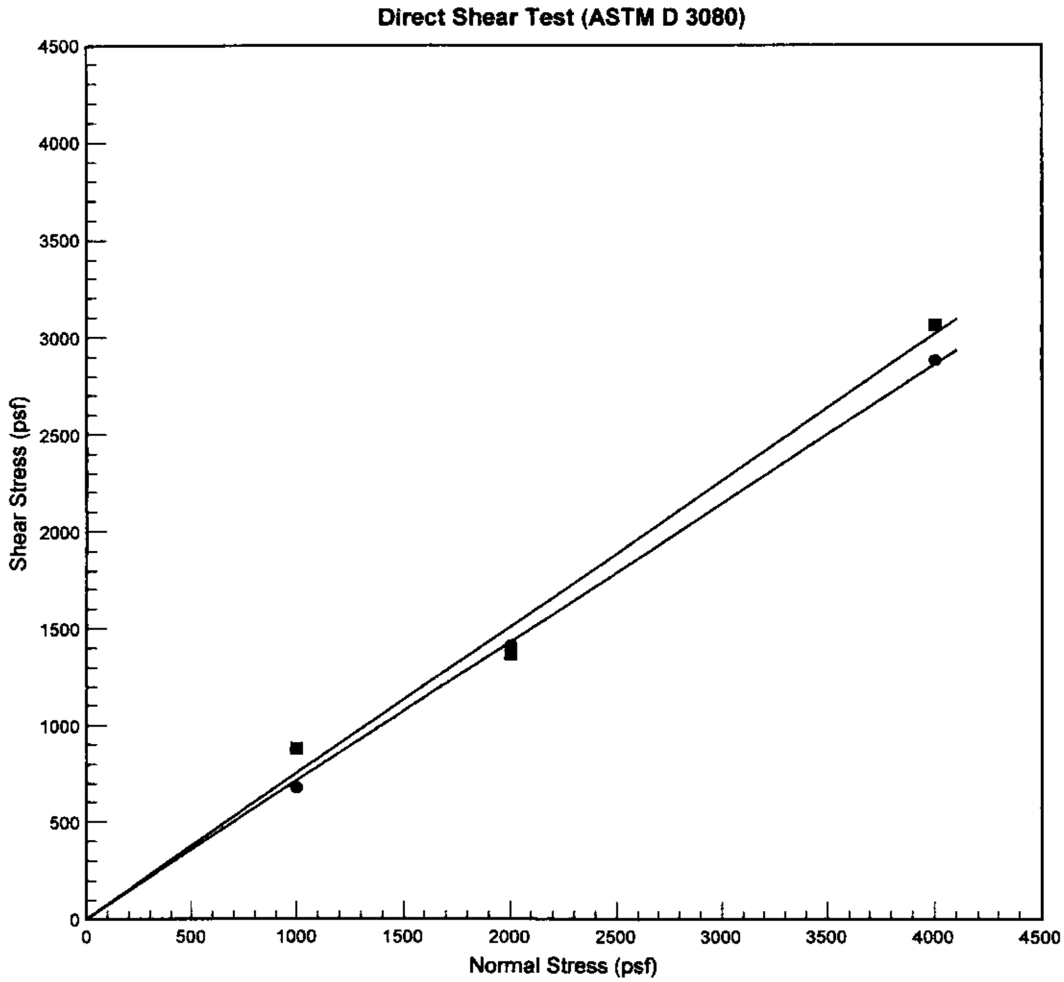


	Boring #	Depth(ft)	Soil/Sample Type	γ_{max} (pcf)	w_{opt} (%)
●	2	0	(SM) Silty Sand, fine to coarse with gravel to 2"	10.0	124.5
■	3	0	(SP-SM) Sand, fine to medium with silt	10.5	113.5

C.H.J. Incorporated

COMPACTION TESTS

Project:	Solar One Demonstration Site		
Location:	Newberry Springs/Ludlow Area, San Bernardino County, CA		
Job No.:	V06100-3	Enclosure:	C-3



	Boring #	Depth(ft)	Soil/Sample Type	γ_d (pcf)	MC(%)	C (psf)	φ(°)
●	2	0	(SM) Silty Sand, fine to coarse with gravel to 2"	112	10.5	0	35.5
■	3	0	(SP-SM Sand, fine to medium with silt	103	11.0	0	37.0

C.H.J. Incorporated

DIRECT SHEAR TESTS

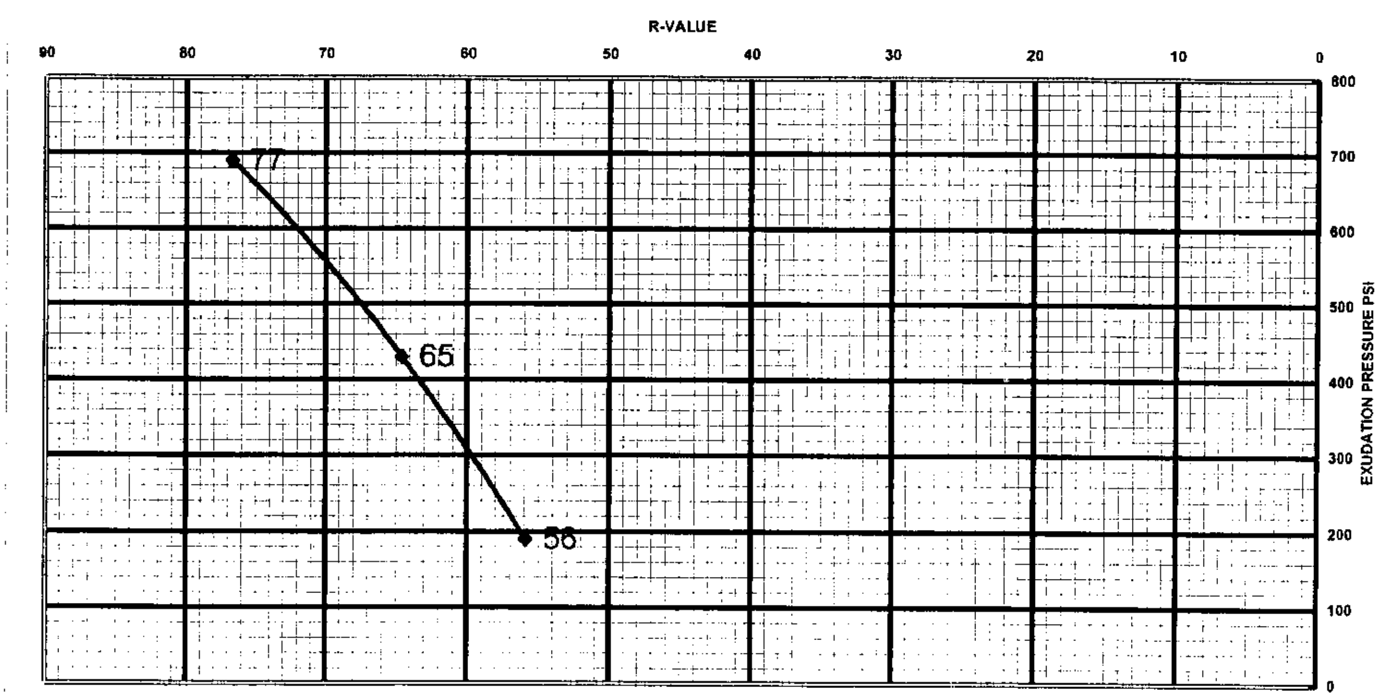
Project:	Solar One Demonstration Site		
Location:	Newberry Springs/Ludlow Area, San Bernardino County, CA		
Job No.:	V06100-3	Enclosure:	C-4

LABORATORY RECORD OF TESTS MADE ON BASE, SUBBASE, AND BASEMENT SOILS

CLIENT: Stirling Energy Sys.
PROJECT:
LOCATION: Pisgah Crater Rd.
R-VALUE # : #1
T.I. : 5.0

	A	B	C	D
COMPACTOR AIR PRESSURE P.S.I.	350	350	350	
INITIAL MOISTURE %	2.7	2.7	2.7	
WATER ADDED, ML	40	50	60	
WATER ADDED %	3.4	4.3	5.1	
MOISTURE AT COMPACTION %	6.1	7.0	7.8	
HEIGHT OF BRIQUETTE	2.54	2.46	2.55	
WET WEIGHT OF BRIQUETTE	1030	1030	1030	
DENSITY LB. PER CU.FT.	115.8	118.6	113.5	
STABILOMETER PH AT 1000 LBS.	14	15	26	
2000 LBS.	24	38	48	
DISPLACEMENT	4.30	4.40	4.60	
R-VALUE	77	65	56	
EXUDATION PRESSURE	690	430	190	
THICK. INDICATED BY STAB.	0.37	0.57	0.71	
EXPANSION PRESSURE	0	0	0	
THICK. INDICATED BY E.P.	0.00	0.00	0.00	

EXUDATION CHART



R-Value: 60

SCHIFF ASSOCIATES

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Table 1 - Laboratory Tests on Soil Samples

C.H.J., Inc.
Sterling Energy
Your #V06100-3, SA #06-1711LAB
4-Oct-06

Sample ID			2A @ 1'	4-3 @ 12'
Resistivity		**Units**		
as-received		ohm-cm	840,000	104,000
saturated		ohm-cm	1,320	760
pH			8.0	8.2
Electrical Conductivity		mS/cm	0.39	0.42
Chemical Analyses				
Cations				
calcium	Ca^{2+}	mg/kg	28	81
magnesium	Mg^{2+}	mg/kg	1.6	2.3
sodium	Na^{1+}	mg/kg	457	424
potassium	K^{1+}	mg/kg	8.9	27
Anions				
carbonate	CO_3^{2-}	mg/kg	153	ND
bicarbonate	HCO_3^{1-}	mg/kg	85	436
flouride	F^{1-}	mg/kg	7.1	8.1
chloride	Cl^{1-}	mg/kg	185	61
sulfate	SO_4^{2-}	mg/kg	94	531
phosphate	PO_4^{3-}	mg/kg	ND	1.9
Other Tests				
ammonium	NH_4^{1+}	mg/kg	ND	ND
nitrate	NO_3^{1-}	mg/kg	52.6	ND
sulfide	S^{2-}	qual	na	na
Redox		mV	na	na

Electrical conductivity in millisiemens/cm and chemical analysis were made on a 1:5 soil-to-water extract.
mg/kg = milligrams per kilogram (parts per million) of dry soil.
Redox = oxidation-reduction potential in millivolts
ND = not detected
na = not analyzed

431 West Baseline Road · Claremont, CA 91711
Phone: 909.626.0967 · Fax: 909.626.3316

APPENDIX "D"

DRILLED SHAFT CALCULATIONS

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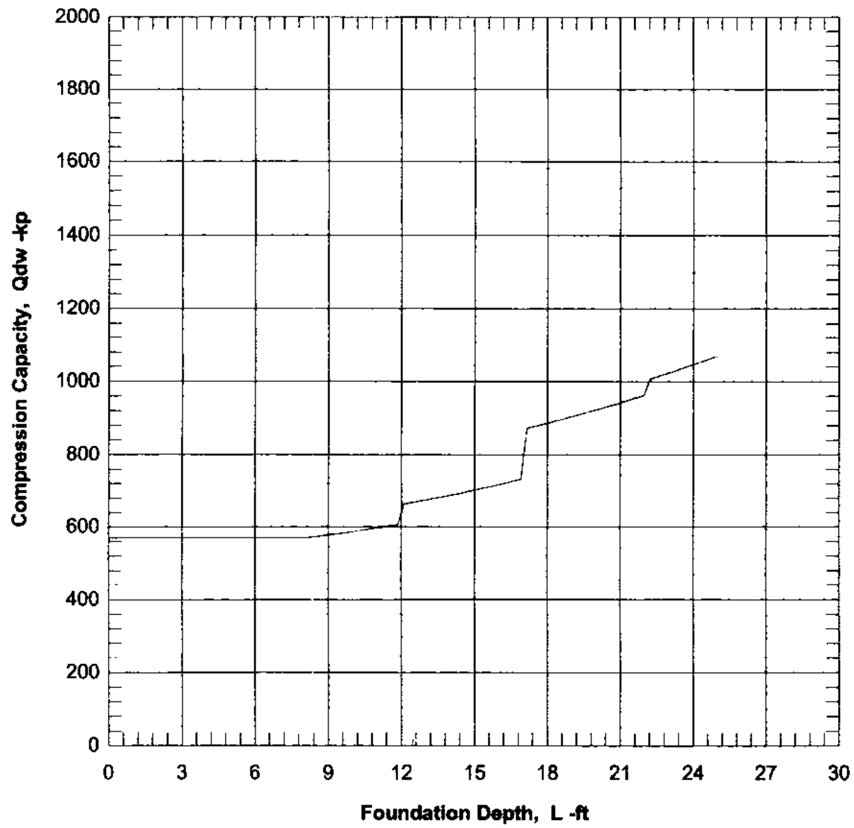
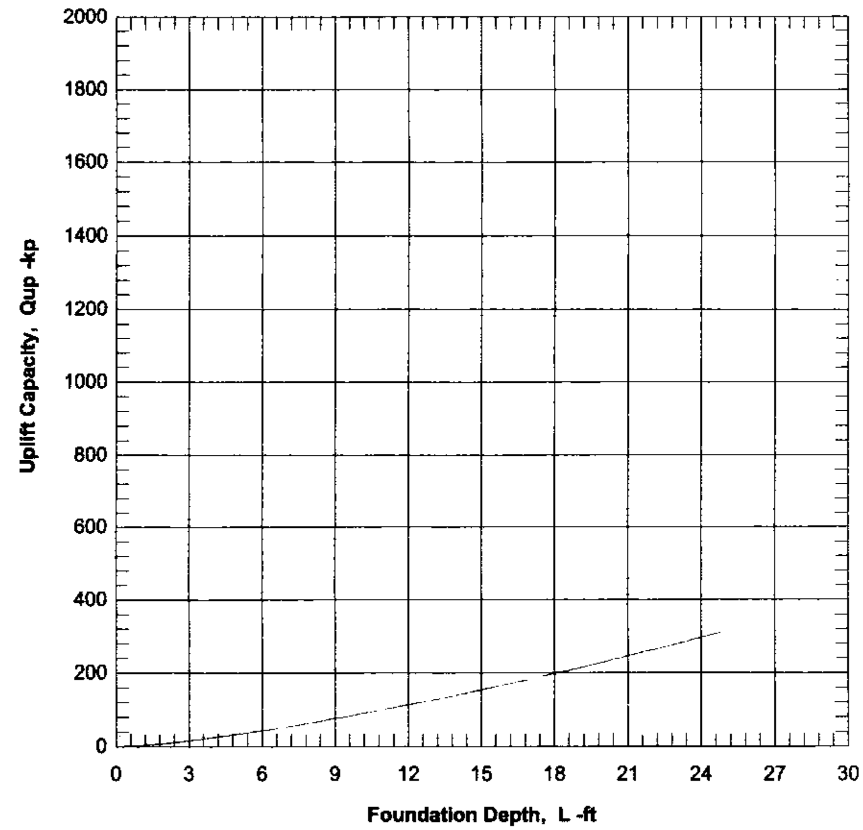
ALLOWABLE CAPACITY vs FOUNDATION DEPTH



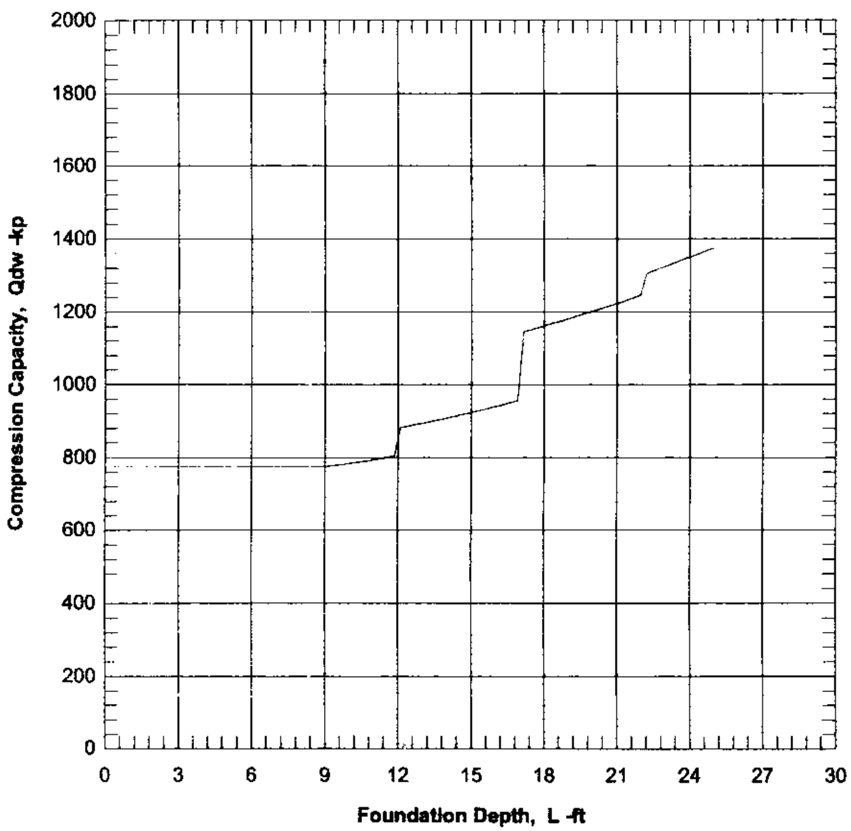


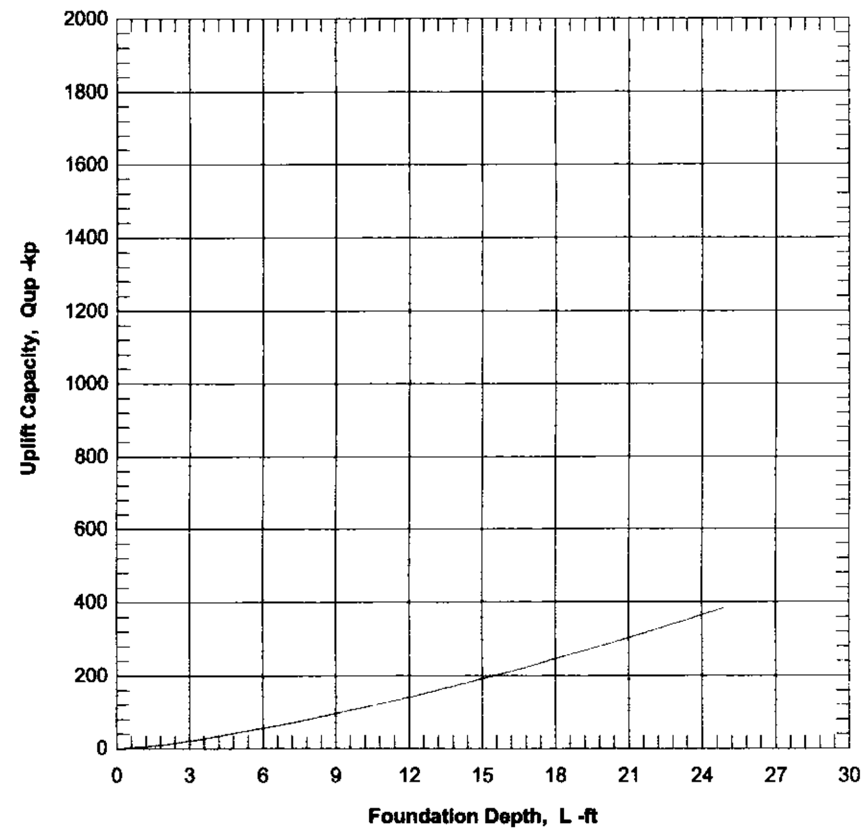
C.H.J. Incorporated

Solar One-Demonstration Site
6ft Caisson

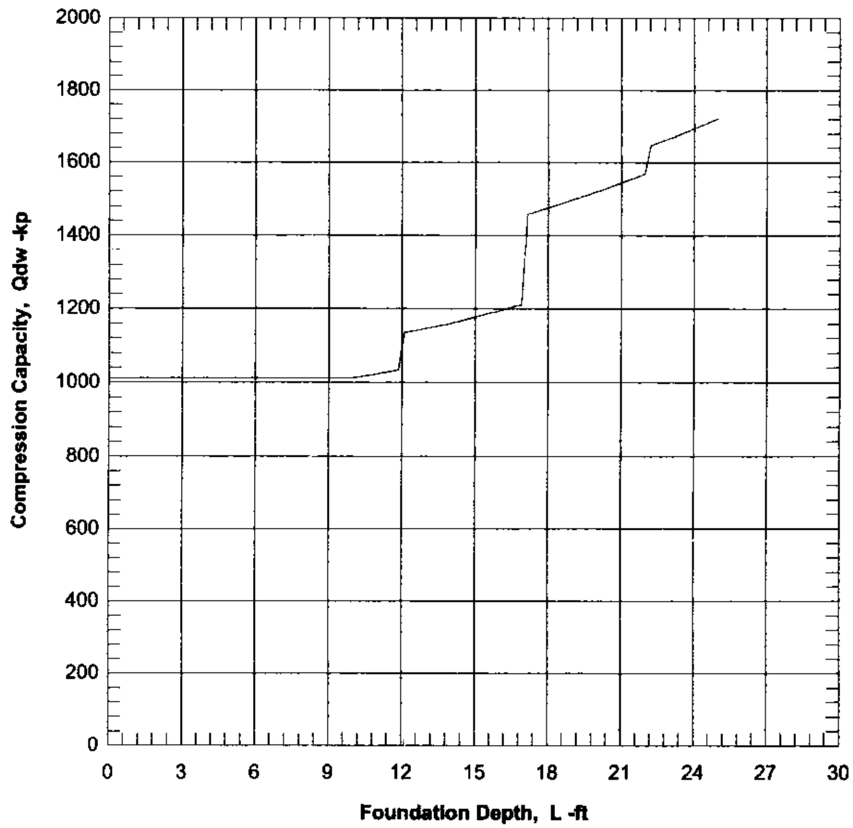
Enclosure "D-1"
Job No. V06100-3

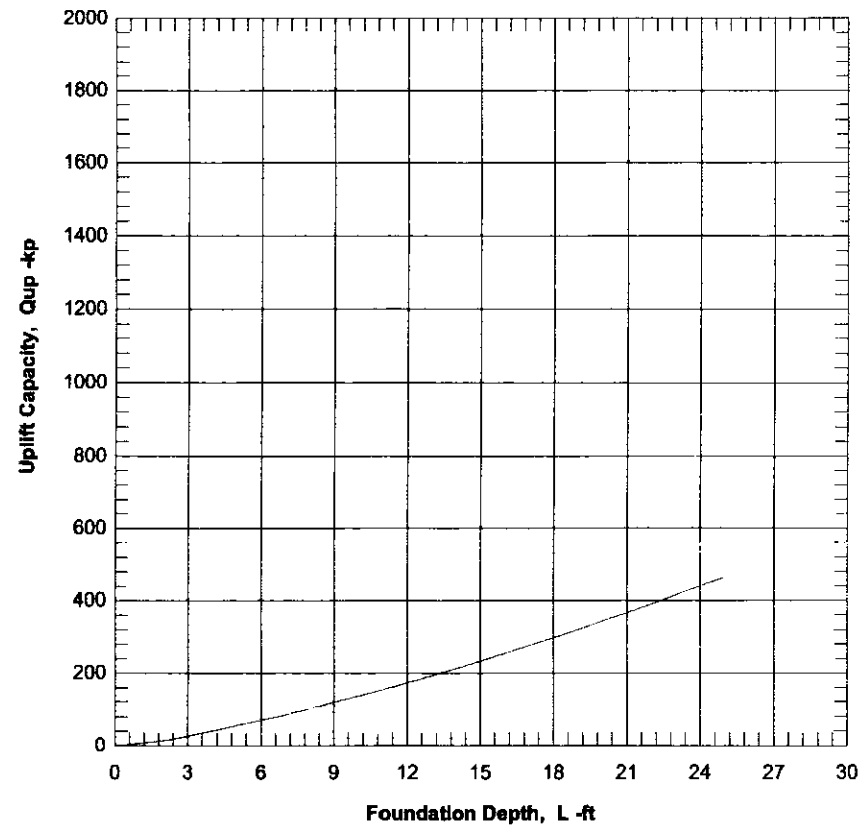
ALLOWABLE CAPACITY vs FOUNDATION DEPTH





ALLOWABLE CAPACITY vs FOUNDATION DEPTH





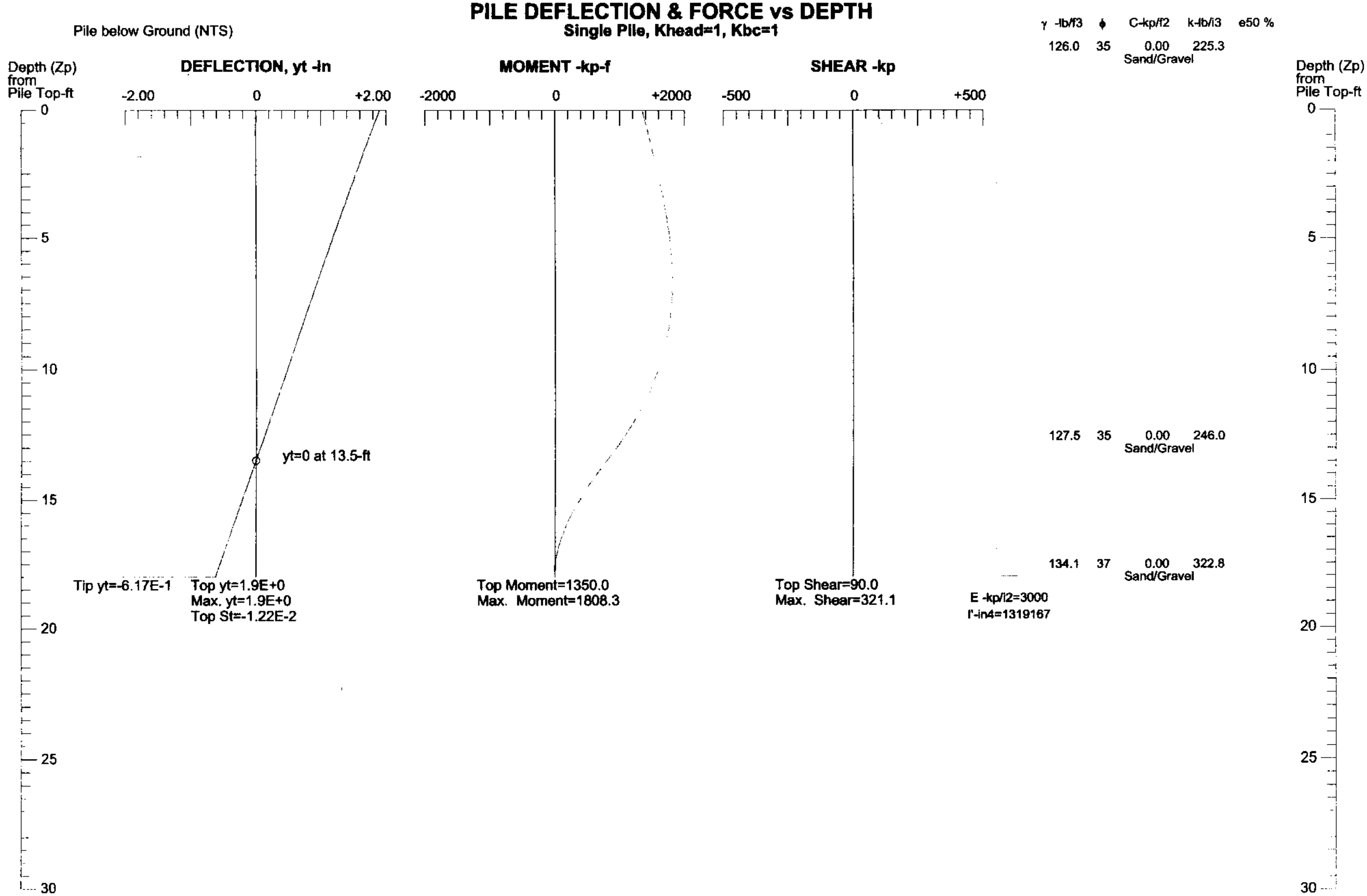
C.H.J. Incorporated

Solar One-Demonstration Site
8ft Caisson

Enclosure "D-3"
Job No. V06100-3

PILE DEFLECTION & FORCE vs DEPTH

Single Pile, Khead=1, Kbc=1

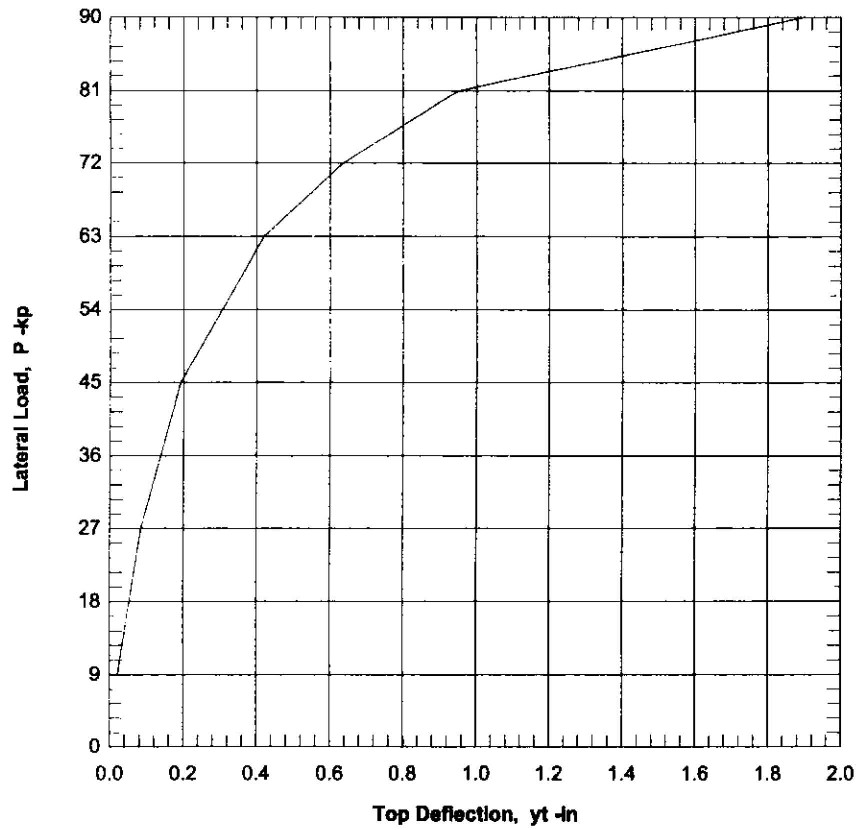


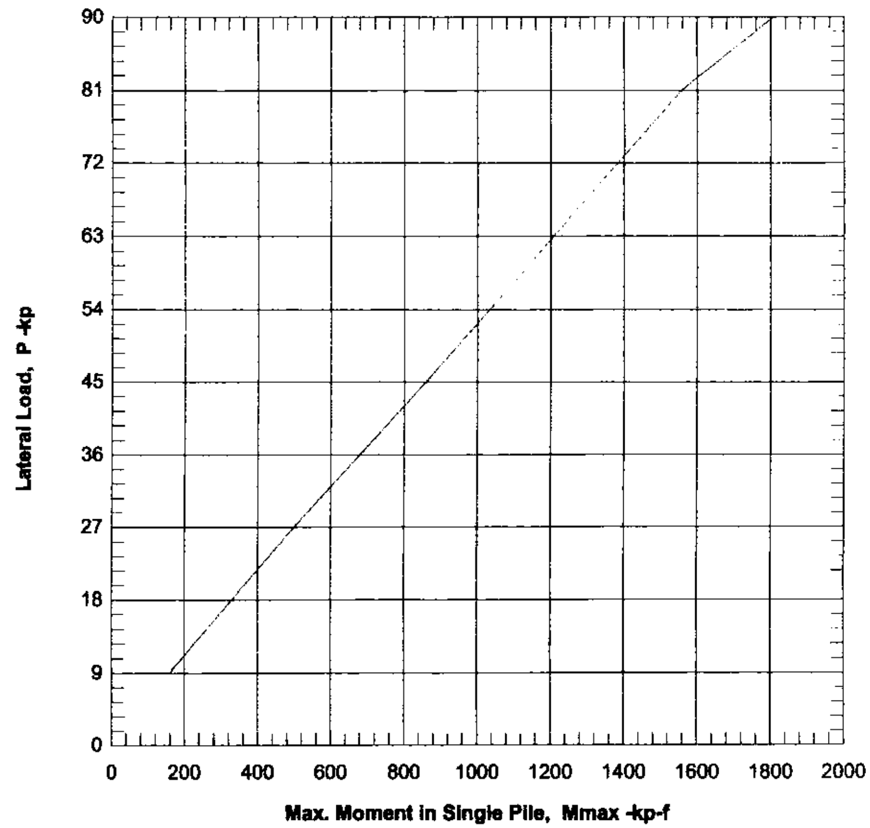
C.H.J. Incorporated

Solar One-Demonstration Site
6ft Caisson

Enclosure "D-4"
Job No. V06100-3

LATERAL LOAD vs DEFLECTION & MAX. MOMENT



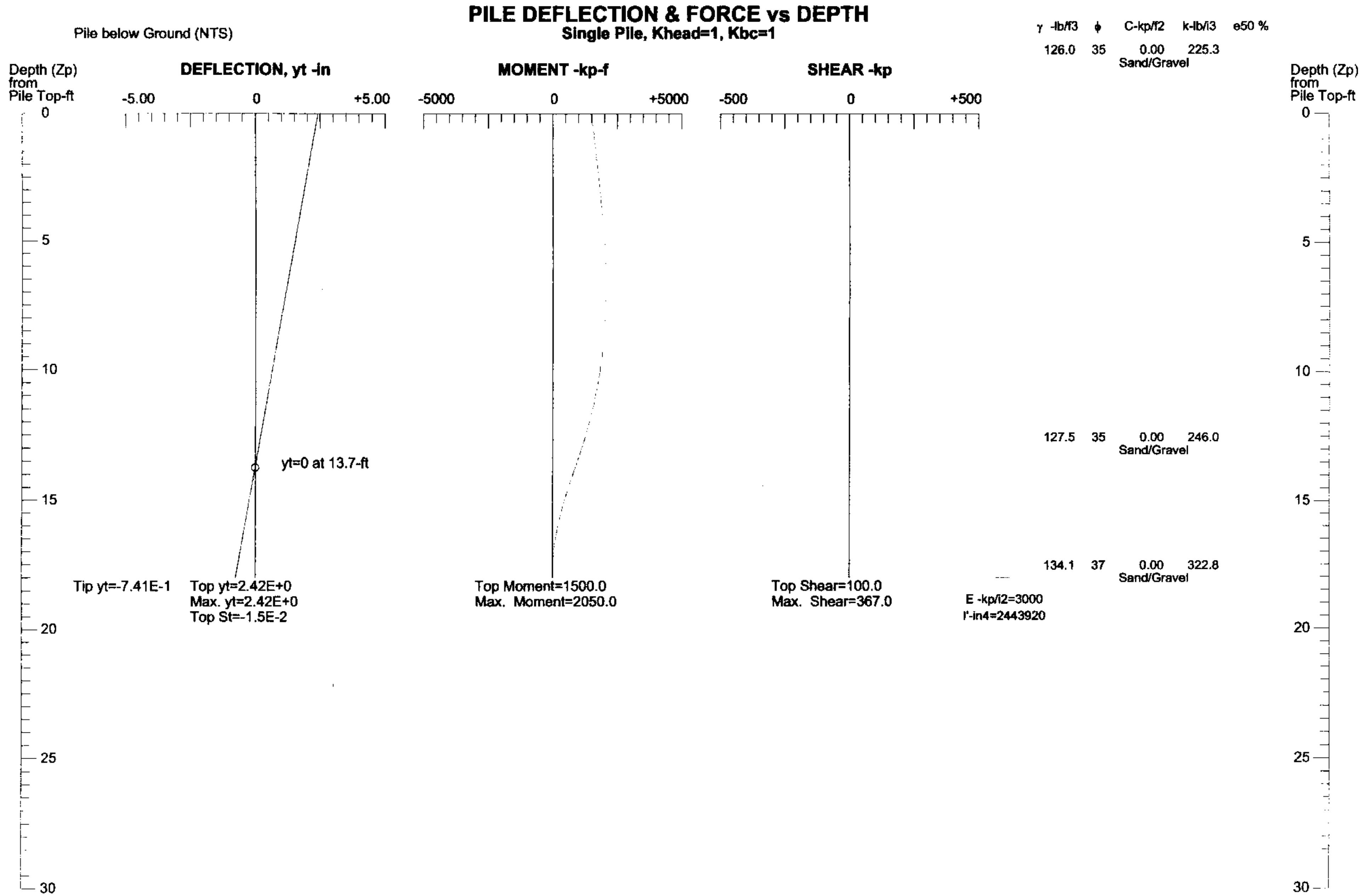


C.H.J. Incorporated

Solar One-Demonstration Site
6ft Caisson

Enclosure "D-5"
Job No. V06100-3

PILE DEFLECTION & FORCE vs DEPTH

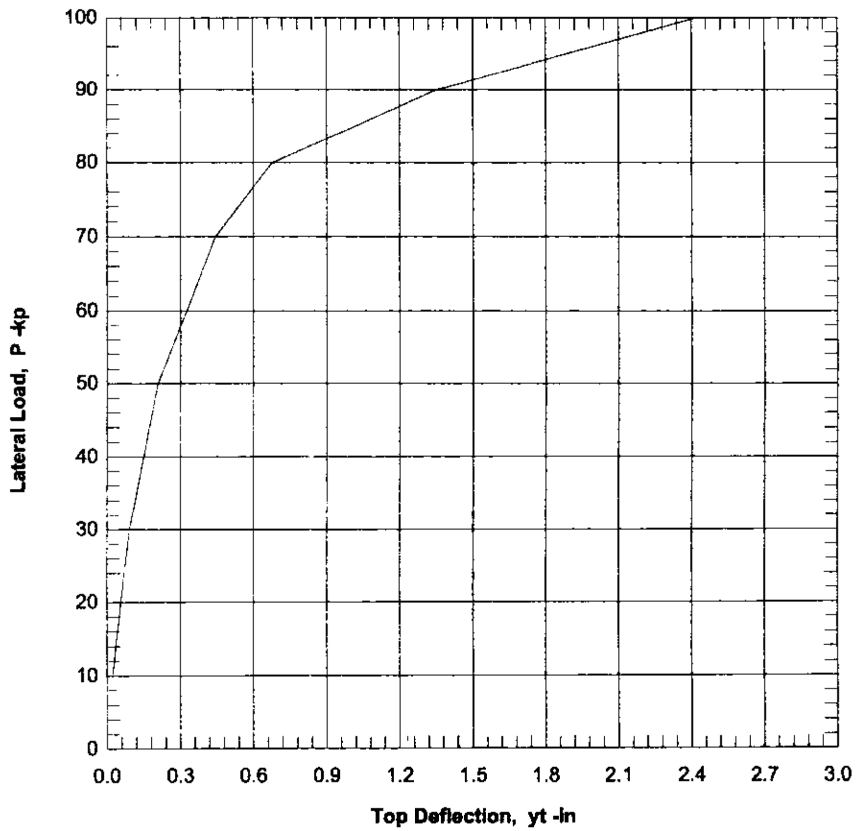


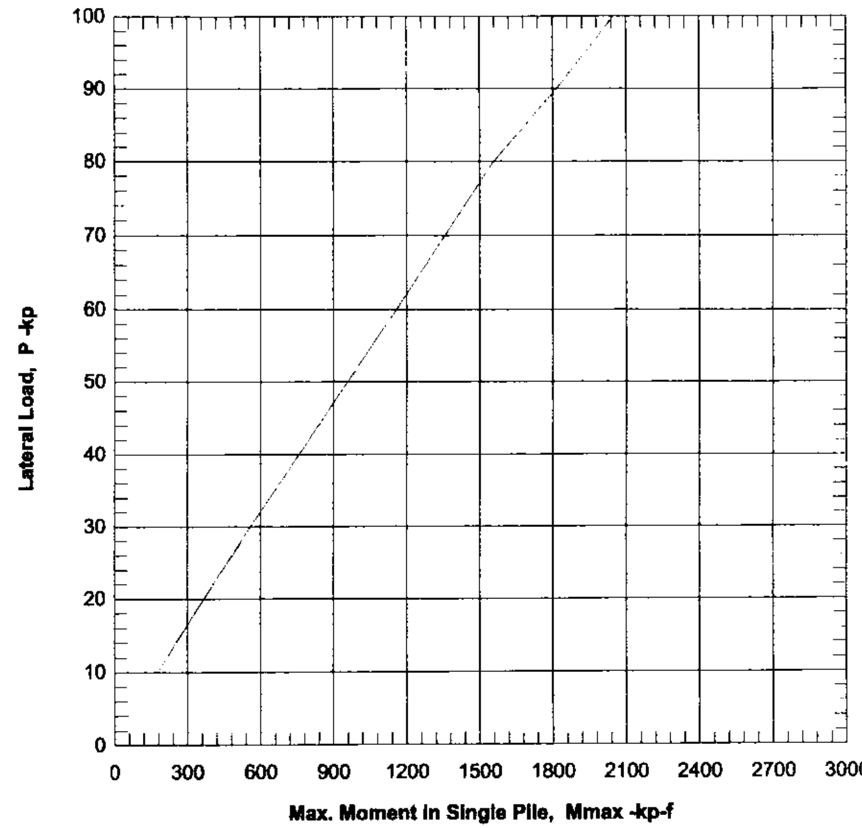
C.H.J. Incorporated

Solar One-Demonstration Site
7ft Caisson

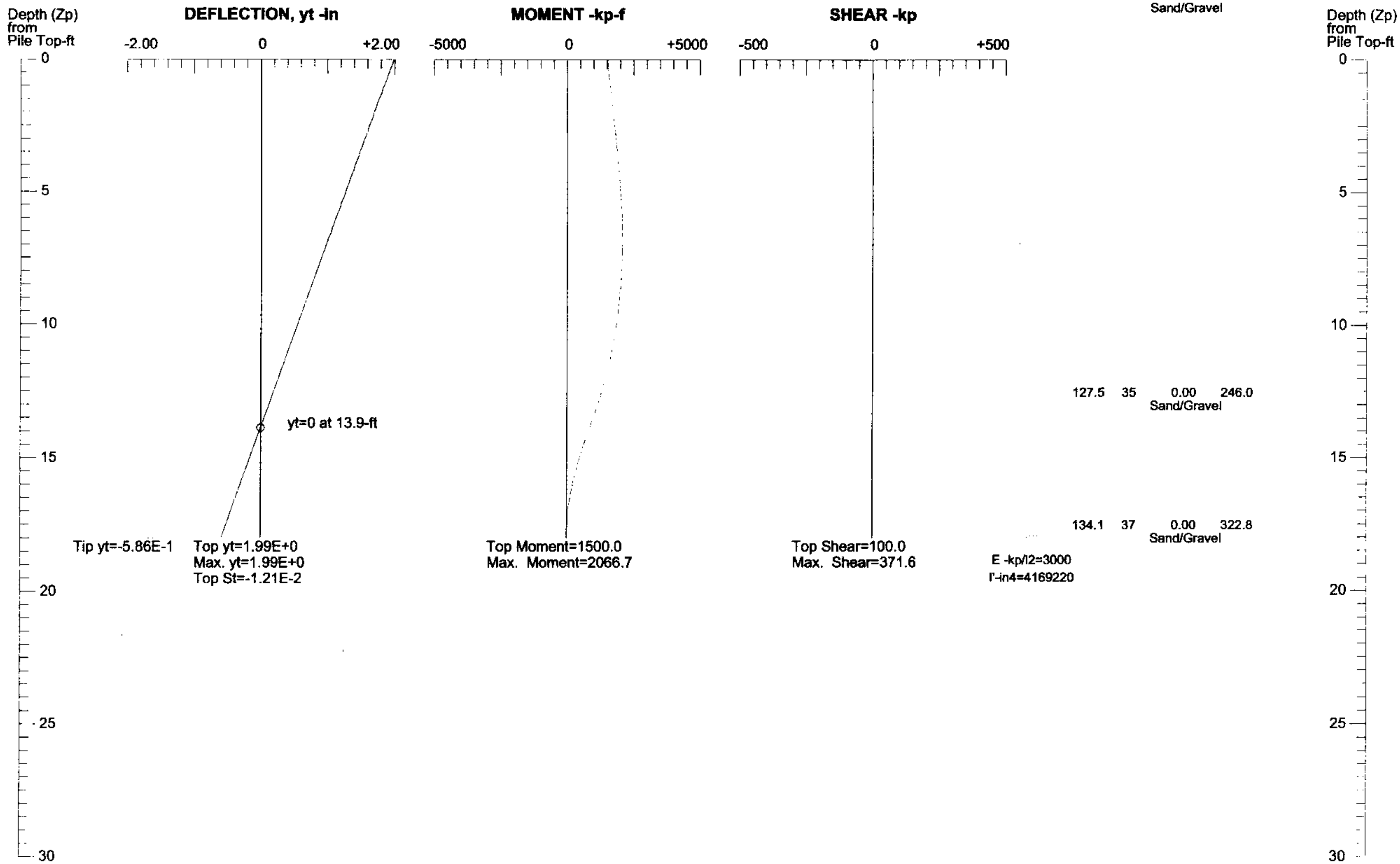
Enclosure "D-6"
Job No. V06100-3

LATERAL LOAD vs DEFLECTION & MAX. MOMENT





PILE DEFLECTION & FORCE vs DEPTH
Single Pile, Khead=1, Kbc=1

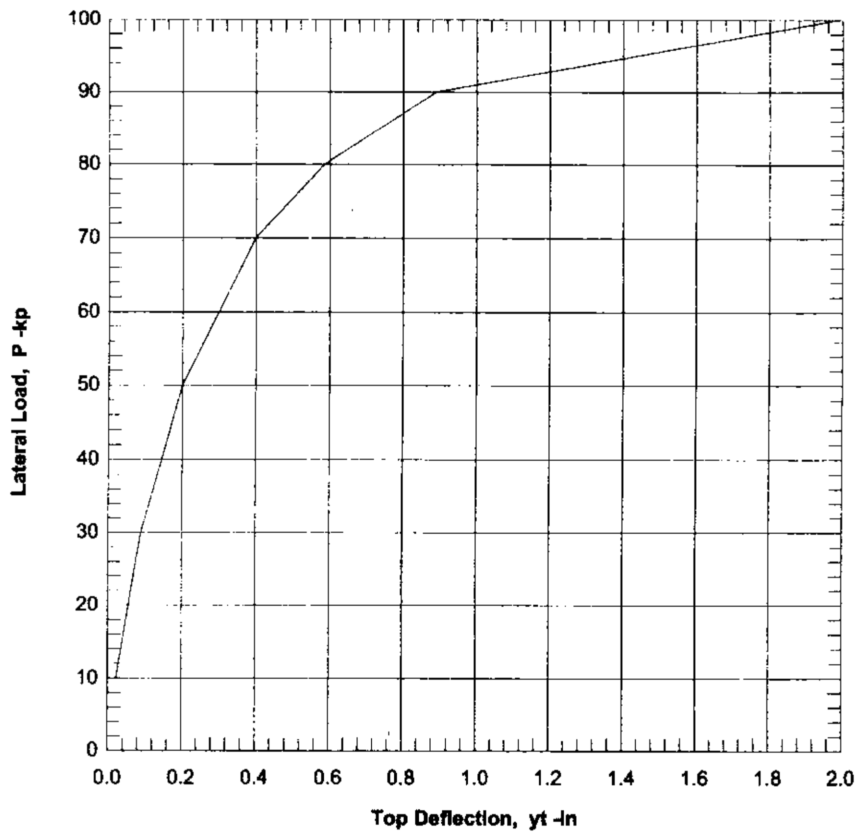


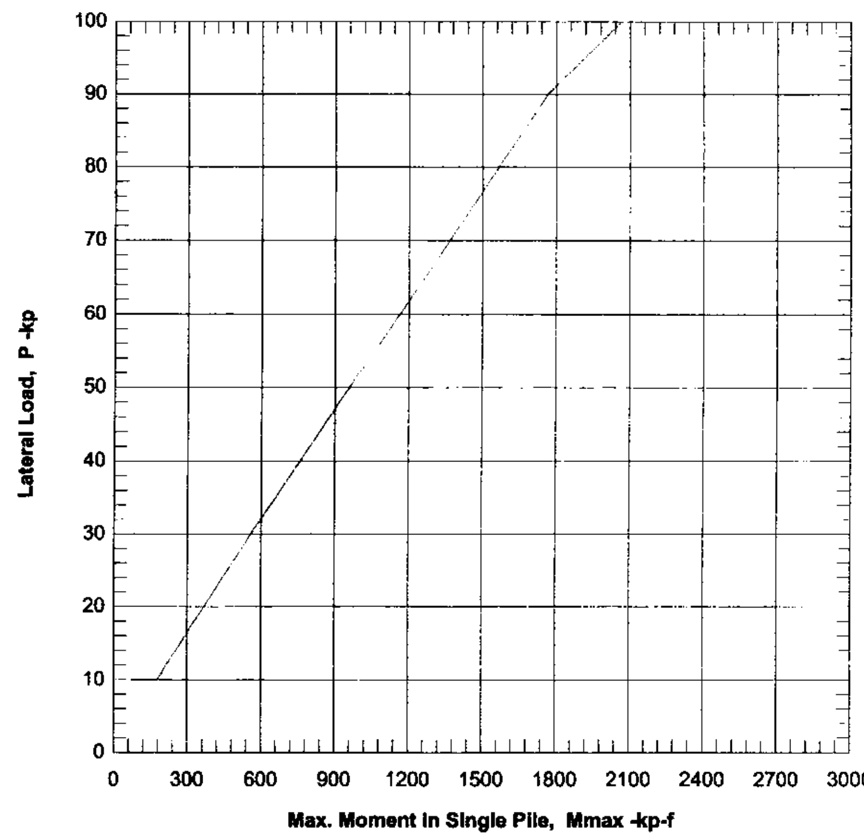
C.H.J. Incorporated

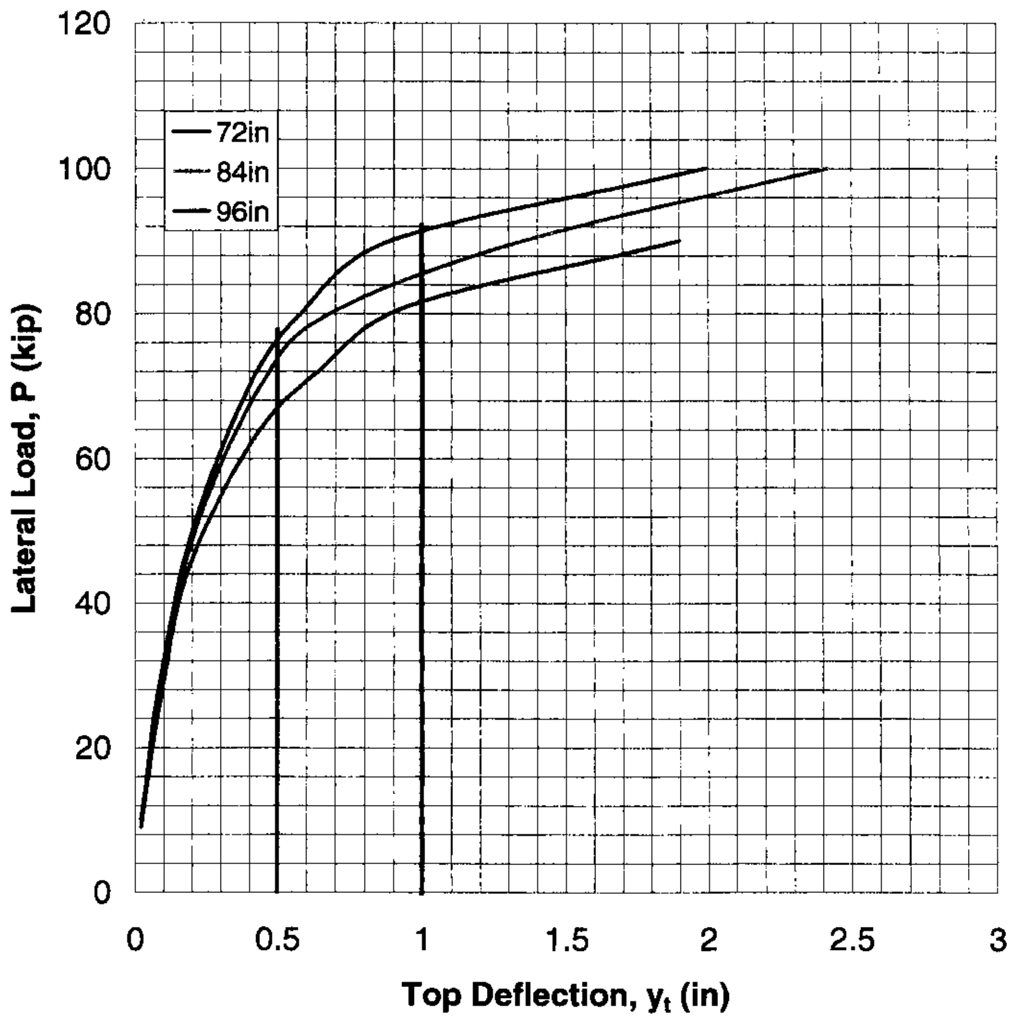
Solar One-Demonstration Site
8ft Caisson

Enclosure "D-8"
Job No. V06100-3

LATERAL LOAD vs DEFLECTION & MAX. MOMENT







C.H.J. Incorporated

Top Deflection - Load Relationships			
Project:	Proposed Solar One-Demonstration Site		
Location:	Newberry Springs/Ludlow Area, San Bernardino, CA		
Job No.:	V06100-3	Enclosure:	D-10

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URS geologists performed a preliminary geologic reconnaissance and field mapping of the proposed project site from October 28 through October 31, 2008. The geologic reconnaissance and mapping activities included general surficial mapping of the contacts between geologic units, measuring and recording structural data, and mapping of the major washes. Due to the size of the site and the preliminary scope of the evaluation, detailed mapping was only performed at selected locations. Selected photographs taken during the performance of the geologic field activities are presented in the photograph log below. Photograph locations are shown on Figure 5.

GEOLOGIC RECONNAISANCE PHOTOGRAPH LOG

Photograph #1

Date: 10/28/08

Comments: Qf – *Quaternary alluvial fan gravel*. Typically light reddish brown Gravelly (~30%) coarse to fine Sand (~50%), with Cobbles (~20%). Granitic and volcanic clasts up to 18”, sub-angular to sub-round and moderately weathered. This formation type typically found in the north end (upper elevations) of the project site.

<table>
<tr>
<td></td>
<td>

Photograph #2

Date: 10/28/08

Comments: Qa – *Quaternary alluvium*. Typically light reddish brown to light brown Gravelly (~15%) fine to coarse Sand (~85%), trace Cobbles. Granitic and volcanic clasts up to 8”, sub-angular to sub-round, moderately weathered, poorly to moderately consolidated. This formation type typically found in the central portion of the site.

</td>
</tr>
<tr>
<td></td>
<td>

Photograph #3

Date: 10/30/08

Comments:
View looking northeast showing **Qa**-*Quaternary Alluvium on the central portion of the site*.

</td>
</tr>
</table>

Photograph #4

Date: 10/30/08

Comments: Qa – *Quaternary alluvium with eolian sand* observed on the surface. The wind blown sand was predominantly observed on the central and southern portions of the project site as a veneer overlying the mapped formations.

Photograph #5

Date: 10/30/08

Comments: Qlc – *Quaternary lacustrine deposits*. These fine-grain dry lake bed deposits were observed on the western portion of the project site. Mud cracks were apparent in localized surface depressions and in the low lying areas of the lacustrine environment.

Photograph #6

Date: 10/30/08

Comments: Qlc – *Quaternary lacustrine deposits*. View of the lacustrine deposits on the western portion of the project site looking west. Note the development of the immature desert pavement.

Photograph #7

Date: 10/30/08

Comments: Qof – *Quaternary older fanglomerate and gravel*. Typically light reddish brown to light brown Sandy Gravel/Gravelly Sand with few Cobbles. Predominantly volcanic clasts up to 15", sub-angular to sub-round, moderately weathered, poorly to moderately consolidated. This formation type typically found in the southern central portion of the site.

Photograph #8

Date: 10/29/08

Comments: ***Qof*** *- Quaternary older fanglomerate and gravel*. Photograph taken while standing on Qof looking north.

Photograph #9

Date: 10/30/08

Comments: ***Qb*** *– Quaternary basalt of the Pisgah flow*. Dark gray Basalt, vesicular, moderately weathered and strong.

Photograph #10

Date: 10/30/08

Comments:
Photograph showing the contact between the Quaternary basalt Pisgah flow and the Quaternary lacustrine deposits. Note the mature desert pavement overlying the lacustrine deposits and the eolian sands partially enveloping the volcanics.

Photograph #11

Date: 10/30/08

Comments:
Photograph looking northwest showing surface expression of the Pisgah fault. This fault feature is located on the western edge of the project site.

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As part of the preliminary geologic reconnaissance and field mapping for the proposed project site, URS geologists performed two surface transect evaluations. The surface transect evaluations were designed and conducted to gain an understanding of the surface geology with respect to grain/clast size distribution. The grain/clast size distribution may be an important factor in the design and installation of the proposed metal fin-pipe foundations used to support the SunCatchers.

The surface transects were performed by first identifying two separate linear routes through relatively undisturbed segments of the geologic units found on the proposed project site. After the transect routes were selected, discrete locations were plotted along each transect, typically 1,500 to 1,800 feet apart. At each location, the URS geologists acquired their position using a handheld global positioning system (GPS), characterized the surface geology with respect to grain/clast size distribution and photographed the location. The surface geology was characterized by observing and recording color, grain sizes and relative percentage present, grain/clast shape, degree of weathering and general lithology.

Transect No. 1 was performed along the eastern boundary of the proposed project site in a southwesterly direction, generally perpendicular to the geologic contacts on the alluvial fan. Transect No. 2 was performed near the northwestern boundary of the proposed project site in a southerly direction, also generally perpendicular to geologic contacts on the alluvial fan. Figure 5 shows the two transect lines and selected evaluation locations. Photographs of each transect and their respective evaluation locations along with detailed geologic observations are presented in the photograph log on the following pages. The grain size distributions are summarized in Table 3 of the report.

SURFACE TRANSECT PHOTOGRAPH LOG
TRANSECT 1

	**Photograph #1** **Date: 10/28/08** **Comments:** **<u>Transect 1, Location 1</u>** Light reddish brown Gravelly (40%) coarse to fine Sand (40%), with Cobbles (20%). Granitic and volcanic clasts up to 18", sub-angular to sub-round, moderately weathered.
	**Photograph #2** **Date: 10/28/08** **Comments:** **<u>Transect 1, Location 2</u>** Light reddish brown Gravelly (40%) coarse to fine Sand (40%), with Cobbles (20%). Granitic and volcanic clasts up to 18", sub-angular to sub-round, moderately weathered.

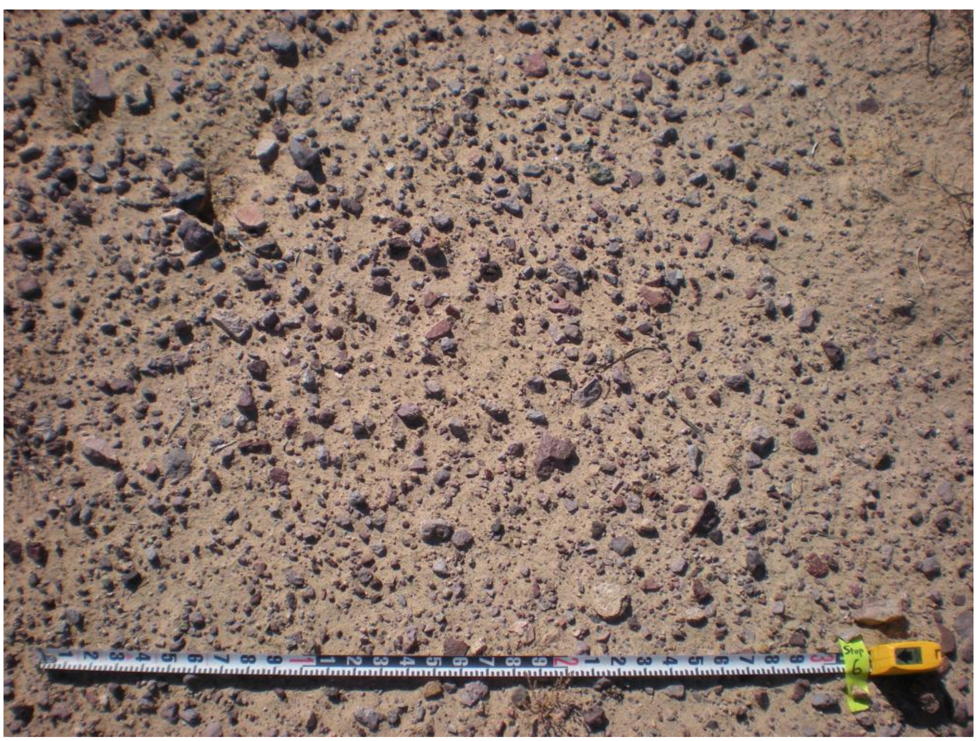
Photograph #3

Date: 10/28/08

Comments:

<u>Transect 1, Location 3</u>

Light reddish brown Gravelly (30%) coarse to fine Sand (60%), with Cobbles (10%). Granitic and volcanic clasts up to 12", sub-angular to sub-round, moderately weathered.



Photograph #4

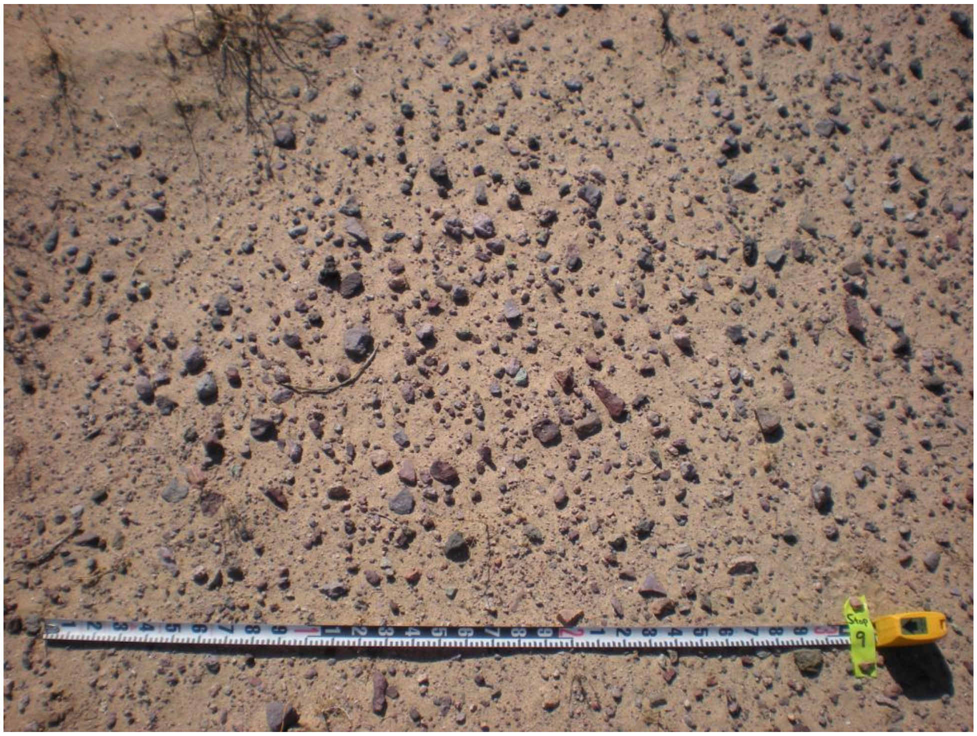
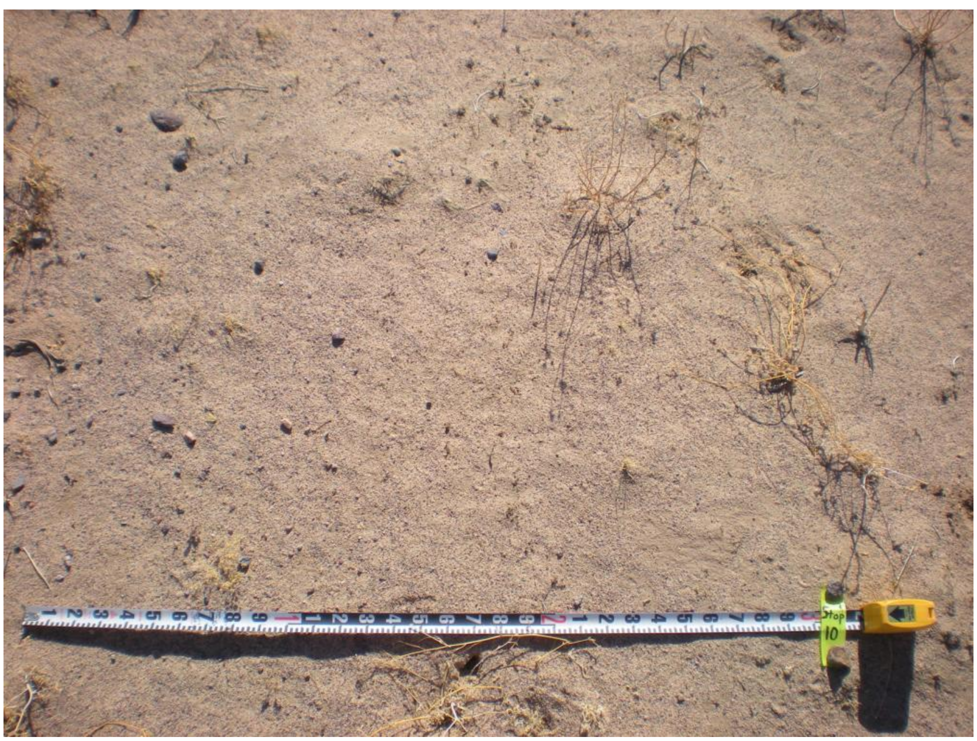
Date: 10/28/08

Comments:

<u>Transect 1, Location 4</u>

Light reddish brown Gravelly (25%) coarse to fine Sand (70%), few Cobbles (5%). Granitic and volcanic clasts up to 12", sub-angular to sub-round, moderately weathered.

	**Photograph #5** **Date: 10/28/08** **Comments:** **<u>Transect 1, Location 5</u>** Light reddish brown Gravelly (20%) fine to coarse Sand (75%), few Cobbles (5%). Granitic and volcanic clasts up to 8”, sub-angular to sub-round, moderately weathered.
	**Photograph #6** **Date: 10/28/08** **Comments:** **<u>Transect 1, Location 6</u>** Light reddish brown Gravelly (15%) fine to coarse Sand (80%), few Cobbles (5%). Granitic and volcanic clasts up to 8”, sub-angular to sub-round, moderately weathered.

	**Photograph #7** **Date: 10/28/08** **Comments:** **<u>Transect 1, Location 7</u>** Light reddish brown fine to coarse Sand (95%), few Gravels (5%), trace Cobbles. Granitic and volcanic clasts up to 6”, sub-angular to sub-round, moderately weathered.
	**Photograph #8** **Date: 10/28/08** **Comments:** **<u>Transect 1, Location 8</u>** Light brown silty fine to coarse Sand (95% including eolian sand), few Gravels (5%). Granitic and volcanic clasts up to 3”, sub-angular to sub-round, moderately weathered.

	**Photograph #9** **Date: 10/28/08** **Comments:** **<u>Transect 1, Location 9</u>** Light brown silty fine to coarse Sand (95% including eolian sand), few Gravels (5%). Granitic and volcanic clasts up to 3”, sub-angular to sub-round, moderately weathered.
	**Photograph #10** **Date: 10/28/08** **Comments:** **<u>Transect 1, Location 10</u>** Dark gray Basalt with veneer of eolian Sand, moderately weathered.

TRANSECT 2

Photograph #1

Date: 10/30/08

Comments:

Transect 2, Location 1

Light brown Gravelly (20%) fine to coarse Sand (80%), trace Cobbles. Granitic and volcanic clasts up to 8”, sub-angular to sub-round, moderately weathered, poorly to moderately consolidated.

Photograph #2

Date: 10/30/08

Comments:

Transect 2, Location 1

View looking northeast showing **Qa-***Quaternary Alluvium*.

Photograph #3

Date: 10/30/08

Comments:
<u>Transect 2, Location 2</u>
Light brown Gravelly (10%) fine to coarse Sand (90%), trace Cobbles. Granitic and volcanic clasts up to 8”, sub-angular to sub-round, moderately weathered, poorly to moderately consolidated.

Photograph #4

Date: 10/30/08

Comments:
<u>Transect 2, Location 2</u>
View looking north showing **Qa**-*Quaternary Alluvium*.

	**Photograph #5** **Date: 10/30/08** **Comments:** **<u>Transect 2, Location 3</u>** Light brown Gravelly (10%) fine to coarse Sand (90% including eolian sand), trace Cobbles. Granitic and volcanic clasts up to 8”, sub-angular to sub-round, moderately weathered, poorly to moderately consolidated.
	**Photograph #6** **Date: 10/30/08** **Comments:** **<u>Transect 2, Location 3</u>** View looking northwest showing **Qa-***Quaternary Alluvium with eolian sand* on the surface.

Photograph #7

Date: 10/30/08

Comments:
Transect 2, Location 4
Light brown Gravelly (20%) fine to coarse Sand (80% including eolian sand), trace Cobbles. Granitic and volcanic clasts up to 8”, sub-angular to sub-round, moderately weathered, poorly to moderately consolidated.

Photograph #8

Date: 10/30/08

Comments:
Transect 2, Location 4
View looking west showing **Qa**-*Quaternary Alluvium with eolian sand* on the surface.

Photograph #9

Date: 10/30/08

Comments:
<u>Transect 2, Location 5</u>
Light brown fine to coarse Sand (95% including eolian sand) with Gravel (5%), trace Cobbles. Granitic and volcanic clasts up to 8", sub-angular to sub-round, moderately weathered, poorly to moderately consolidated.

Photograph #10

Date: 10/30/08

Comments:
<u>Transect 2, Location 5</u>
View looking northeast showing **Qa**-*Quaternary Alluvium with eolian sand* on the surface. Incised washes become braided streams.

	**Photograph #11** **Date: 10/30/08** **Comments:** **<u>Transect 2, Location 6</u>** Light brown fine to medium eolian Sand, underlain by reddish brown, well consolidated, fine to coarse Sand (95%), few Gravels (5%). Granitic and volcanic clasts up to 4", sub-angular to sub-round, moderately weathered.
	**Photograph #12** **Date: 10/30/08** **Comments:** **<u>Transect 2, Location 6</u>** View looking northeast showing **Qa**-*Quaternary Alluvium with eolian sand* on the surface. Braided streams are apparent.